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Federal Highway Administration > Publications > Research Publications > Safety > Human Factors > 01103 > Highway Design Handbook for Older Drivers and Pedestrians

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Highway Design Handbook for Older Drivers and Pedestrians

I. INTERSECTIONS (AT-GRADE)

RATIONALE AND SUPPORTING EVIDENCE

This section of the Handbook is organized in terms of the same classes of highway features as the Recommendations: I. Intersections (At-Grade), II. Interchanges (Grade Separation), III. Roadway Curvature and Passing Zones, IV. Construction/Work Zones, and V. Highway-Rail Grade Crossings (Passive). Within each of these five classes, subsections are organized in terms of design elements with unique geometric, operational, and/or traffic control characteristics, also consistent with the recommendations.

At the beginning of each subsection within a class of highway features, reference material for a particular design element is introduced using a cross-reference table. This table relates the discussion in that subsection--as well as the associated recommendations, presented earlier--to entries in standard reference manuals consulted by practitioners in this area. Principal among these reference manuals are the *Manual on Uniform Traffic Control Devices* (Federal Highway Administration [FHWA], 2000); the *Policy on Geometric Design of Highways and Streets* [the Green Book] (American Association of State Highway and Transportation Officials [AASHTO], 1994); and the *Traffic Engineering Handbook* (ITE, 1999). Other standard references with more restricted applicability, which also appear in the cross-reference tables for selected design elements, include the National Cooperative Highway Research Program (NCHRP) Report No. 279, *Intersection Channelization Design Guide* (Neuman, 1985); *Roundabouts: An Informational Guide* (FHWA, 2000); the *Roadway Lighting Handbook* (FHWA, 1978); the *Railroad-Highway Grade Crossing Handbook* (FHWA, 1986); and the *Highway Capacity Manual* (TRB, 1998).

Material in this part of the Handbook represents, to as great an extent as possible at the time of its development, the results of empirical work with older driver or pedestrian samples for investigations with the specific highway features of interest. Observational and controlled field studies were given precedence, together with laboratory simulations employing traffic stimuli and relevant situational cues. Crash data are cited as appropriate. In addition, some citations reference studies showing effects of design changes, where the predicted impact on (older) driver performance is tied logically to the results of research on age differences in response capability.

I. INTERSECTIONS (AT-GRADE)

The following discussion presents the rationale and supporting evidence for Handbook recommendations pertaining to these 17 design elements (A-Q):

A. Design Element: Intersecting Angle (Skew)

Table 1. Cross-references of related entries for intersecting angle (skew).

Applications in Standard Reference Manuals			
MUTCD (2000)	**AASHTO Green Book (1994)**	**NCHRP 279 Intersection Channelization Design Guide (1985)**	**Traffic Engineering Handbook (1999)**
Sects. 2B.39 & 4D.17	Pg. 426, Para. 5 Pg. 628, Item C.4 Pg. 630, Para. 1 Pgs. 641-645, Sects. on *Multileg Intersections* & *Alinement* Pgs. 648-651, Tables IX-1 & IX-2 Pgs. 663-664, Sect. on *Oblique-Angle Turns* Pg. 673, Para. 5 Pgs. 676-680, Sects. *on Divisional Islands, Refuge Islands,* & *Island Size and Designation* Fig. IX-23 Pgs. 689-690, Sect. on *Oblique-Angle Turns with Corner Islands* Pg. 691, Table IX-4 Pgs. 718-720, Sect. on *Effect of Skew* Pgs. 764-767, Sect. on *Effect of Skew*	Pg. 19, Top fig. Pg. 21, Item 5 Pg. 25, Para. 2 Pg. 30, Para. 1 & top three figs. Pg. 31, Para. 3 & bottom left fig. Pgs. 42-44, Sect. on *Angle of Intersection* Pg. 45, Fig. 4-5 Pg. 71, Top two figs. Pgs. 100-105, Intersct. Nos. 7 -9 Pgs. 148-149, Intersct. No. 35	Pg.384, 5th Principle Pg. 385, Sect. on *Angle of Intersection* Pg. 399, Para. 2 Pg. 435, Para. 4

There is broad agreement that right-angle intersections are the preferred design. Decreasing the angle of the intersection makes detection of and judgments about potential conflicting vehicles on crossing roadways much more difficult. In addition, the amount of time required to maneuver through the intersection increases, for both vehicles and pedestrians, due to the increased pavement area. However, there is some inconsistency among reference sources concerning the degree of skew that can be safely designed into an intersection. The Green Book states that although a right-angle crossing normally is desired, an angle of 60 degrees provides most of the benefits that are obtained with a right-angle intersection. Subsequently, factors to adjust intersection sight distances for skewness are suggested for use only when angles are less than 60 degrees (AASHTO, 1994). However, another source on subdivision street design states that: "Skewed intersections should be avoided, and in no case should the angle be less than 75 degrees" (Institute of Transportation Engineers [ITE], 1984). The *Traffic Engineering Handbook* (ITE, 1999) states that: "Crossing roadways should intersect at 90 degrees if possible, and not less than 75 degrees." It further states that: "Intersections with severe skew angles (e.g., 60 degrees or less) often experience operational or safety problems. Reconstruction of such locations or institution of more positive traffic control such as signalization is often necessary." With regard to intersection design issues on two-lane rural highways, ITE (1999) states that: "Skew angles in excess of 75 degrees often create special problems at stop-controlled rural intersections. The angle complicates the vision triangle for the stopped vehicle; increases the time to cross the through road; and results in a larger, more potentially confusing intersection."

Skewed intersections pose particular problems for older drivers. Many older drivers experience a decline in head and neck mobility, which accompanies advancing age and may contribute to the slowing of psychomotor responses. Joint flexibility, an essential component of driving skill, has been estimated to decline by approximately 25 percent in older adults due to arthritis, calcification of cartilage, and joint deterioration (Smith and Sethi, 1975). A restricted range of motion reduces an older driver's ability to effectively scan to the rear and sides of his or her vehicle to observe blind spots, and similarly may be expected to hinder the timely recognition of conflicts during turning and merging maneuvers at intersections (Ostrow, Shaffron, and McPherson, 1992). For older drivers, diminished physical capabilities may affect their performance at intersections designed with acute angles by requiring them to turn their heads further than would be required at a right-angle intersection. This obviously creates more of a problem in determining appropriate gaps. For older pedestrians, the longer exposure time within the intersection becomes a major concern.

Isler, Parsonson, and Hansson (1997) measured the maximum head rotation of 20 drivers in each of four age groups: less than age 30; ages 40 to 59; ages 60 to 69; and age 70 and older, as well as their horizontal peripheral visual field. The oldest subjects exhibited an average decrement of approximately one-third of head range of movement compared with the youngest group of subjects. The mean maximum head movement (in one direction) was 86 degrees for the youngest drivers, 72 degrees for drivers ages 40 to 59, 67 degrees for drivers ages 60 to 69, and 59 degrees for drivers age 70+. In addition, the percentage of drivers with less than 30 degrees of horizontal peripheral vision increased with increases in age, from 15 percent of the younger driver sample to 65 percent of the drivers age 70+. Three of the oldest drivers had less than 50 degrees of head movement and two of these drivers also had less than 20 degrees of horizontal peripheral vision.

In a survey of older drivers conducted by Yee (1985), 35 percent of the respondents reported problems with arthritis and 21 percent indicated difficulty in turning their heads to scan rearward while driving. Excluding vision/visibility problems associated with nighttime operations, difficulty with head turning placed first among *all* concerns mentioned by older drivers participating in a more recent focus group conducted to examine problems in the use of intersections where the approach leg meets the main road at a skewed angle, and/or where channelized right-turn lanes require an exaggerated degree of head/neck rotation to check for traffic conflicts before merging (Staplin, Harkey, Lococo, and Tarawneh, 1997). Comments about this geometry centered around the difficulty older drivers experience turning their heads at angles less than 90 degrees to view traffic on the intersecting roadway, and several participants reported an increasing reliance on outside rearview mirrors when negotiating highly skewed angles. However, they reported that the outside mirror is of no help when the roads meet at the middle angles (e.g., 40 to 55 degrees) and a driver is not flexible enough to physically turn to look for traffic. In an observational field study conducted as a part of the same project, Staplin et al. (1997) found that approximately 30 percent of young/middle-aged drivers (ages 25-45) and young-old drivers (ages 65-74) used their mirrors in addition to making head checks before performing a right-turn-on-red (RTOR) maneuver at a skewed intersection (a channelized right-turn lane at a 65-degree skew). By comparison, none of the drivers age 75 and older used their mirrors; instead, they relied solely on information obtained from head/neck checks. In this same study, it was found that the likelihood of a driver making an RTOR maneuver is reduced by intersection skew angles that make it more difficult for the driver to view conflicting traffic.

The practical consequences of restricted head and neck movement on driving performance at T-intersections were investigated by Hunter-Zaworski (1990), using a simulator to present videorecorded scenes of intersections with various levels of traffic volume and sight distance in a 180-degree field of view from the driver's perspective. Drivers in two subject groups, ages 30-50 and 60-80, depressed a brake pedal to watch a video presentation (on three screens), then released the pedal when it was judged safe to make a left turn; half of *each* age group had a restricted range of neck movement as determined by goniometric measures of maximum (static) head-turn angle. Aside from demonstrating that skewed intersections are hazardous for any driver with an impairment in neck movement, this study found that maneuver decision time increased with both age *and* level of impairment. Thus, the younger drivers in this study were able to compensate for their impairments, but older drivers both with and without impairments were unable to make compensations in their (simulated) intersection response selections.

These research findings reinforce the desirability of providing a 90-degree intersection geometry and endorse the ITE (1984) recommendation establishing a 75-degree minimum as a practice to accommodate age-related performance deficits.

B. Design Element: Receiving Lane (Throat) Width for Turning Operations

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Table 2. Cross-references of related entries for receiving lane (throat) width for turning operations.

Applications in Standard Reference Manuals		
AASHTO Green Book (1994)	**NCHRP 279 Intersection Channelization Design Guide (1985)**	**Traffic Engineering Handbook (1999)**
Pgs. 200-211, Sects. on *Widths for Turning Roadways at Intersections* & *Widths Outside Traveled Way Edges* Pg. 213, Table III-21 Pg. 647, Para. 2 Pg. 673, Para. 5 Pg. 676, Paras. 3-5 Pg. 678, Fig. X-24	Pg. 10, Table 2-4 Pg. 57, Para. 5, 1st Bullet Pg. 58, Fig. 4-20 Pg. 63, Sect. on *Lane Widths* Pg. 69, Sect. on *Width of Roadways* Pg. 73, Fig. 4-29 Pg. 107, Fig. c Pg. 113, Fig. a Pg. 115, Figs. d- e Pg. 120, Item 3 Pg. 122, Item 2 Pg. 125, Intersect. No. 19	Pg. 319, Para. 4 Pg. 386, Para. 5 Pg. 435, Para. 4

Design recommendations for lane width at intersections follow from consideration of vehicle maneuver requirements and their demands on drivers. Positioning a vehicle within the lane in preparation for turning has been rated as a critical task (McKnight and Adams, 1970). Swinging too wide to lengthen the turning radius and minimize rotation of the steering wheel ("buttonhook turn") while turning left or right is a common practice of drivers lacking strength (including older drivers) and physically limited drivers (McKnight and Stewart, 1990).

Two factors can compromise the ability of older drivers to remain within the boundaries of their assigned lanes during a left turn. One factor is the diminishing ability to share attention (i.e., to assimilate and concurrently process multiple sources of information from the driving environment). The other factor involves the ability to turn the steering wheel sharply enough, given the speed at which they are traveling, to remain within the boundaries of their lanes. Some older drivers seek to increase their turning radii by initiating the turn early and rounding-off the turn. The result is either to cut across the apex of the turn, conflicting with vehicles approaching from the left, or to intrude upon a far lane in completing the turn.

Lane widths are addressed in the *Intersection Channelization Design Guide* (Neuman, 1985). A recommendation for (left) turning lanes, which also applies to receiving lanes, is that "3.6-m (12-ft) widths are desirable, (although) lesser widths may function effectively and safely. Absolute minimum widths of 2.7 m (9 ft) should be used only in unusual circumstances, and only on low-speed streets with minor truck volumes." Similarly, the ITE (1984) guidelines suggest a minimum lane width of 3.3 m (11 ft) and specify 3.6 m (12 ft) as desirable. These guidelines suggest that wider lanes be avoided due to the resulting increase in pedestrian crossing distances. However, the ITE guidelines provide a range of lane widths at intersections from 2.7 m to 4.3 m (9 ft to 14 ft), where the wider lanes would be used to accommodate larger turning vehicles, which have turning paths that sweep a path from 4.1 m (13.6 ft) for a single-unit truck or bus, up to 6.3 m (20.6 ft) for a semitrailer. Thus, wider (3.6 m [12 ft]) lanes used to accommodate (right) turning trucks also are expected to benefit (left) turning drivers. Further increases in lane width for accommodation of heavy vehicles may result in unacceptable increases in (older) pedestrian crossing times, however.

Results of field observation studies conducted by Firestine, Hughes, and Natelson (1989) found that trucks performing turns on urban roads encroached into other lanes on streets with widths of less than 3.6 m (12 ft). They noted that on rural roads, lanes wider than 3.6 m or 4.0 m (12 ft or 13 ft) allowed oncoming vehicles on the cross street to move further right to avoid trucks, and shoulders wider than 1.2 m (4 ft) allowed oncoming vehicles a greater margin of safety.

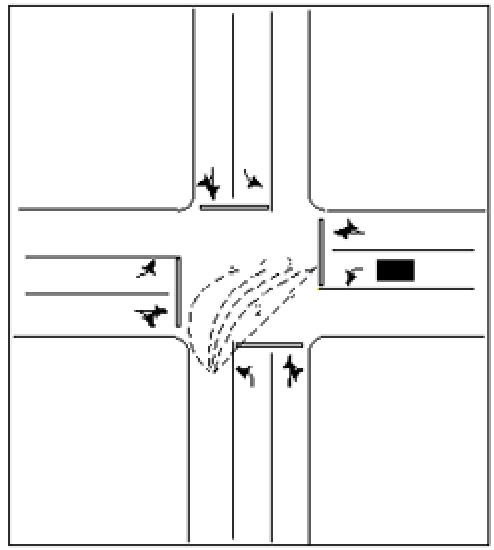


Figure 1. Turning path taken by left-turning vehicles, where 1=encroach into opposing cross-traffic stream; 2, 3, and 4=proper turning from different points within the intersection; and 5=left turn from a position requiring a greater-than-90-degree turn to enter the cross street.

In an observational field study conducted to determine how older drivers (age 65 and older) compare with younger drivers during left-turn operations under varying intersection geometries, one variable that showed significant differences in older and younger driver behavior was turning path (Staplin, Harkey, Lococo, and Tarawneh, 1997). Older drivers encroached into the opposing lane of the cross street (see figure 1, turning path trajectory number 1) when making the left turn more often than younger drivers at the location where the throat width (equivalent to the lane width) measured 3.6 m (12 ft). Where the throat width measured 7 m (23 ft), which consisted of a 3.6-m (12-ft) lane and a 3.3-m (11-ft) shoulder, there was no significant difference in the turning paths. The narrower throat width resulted in higher encroachments by older drivers, who physically may have more difficulties maneuvering their vehicles through smaller areas.

These data sources indicate that a 3.6-m (12-ft) lane width provides the most reasonable tradeoff between the need to accommodate older drivers, as well as larger turning vehicles, without penalizing the older pedestrian in terms of exaggerated crossing distance.

C. Design Element: Channelization

Table 3. Cross-references of related entries for channelization.

Applications in Standard Reference Manuals				
MUTCD (2000)	**AASHTO Green Book (1994)**	**Roadway Lighting Handbook (1978)**	**NCHRP 279 Intersection Channelization Design Guide (1985)**	**Traffic Engineering Handbook (1999)**
Sect. 1A.13, *channelizing line markings* Sect. 3B.03 Sect. 3B.05 Sects. 3B.09, 3B.10, 3B.19,	Pg. 369, Para. 2 Pg. 517, Paras. 5-6 Pg. 518, Fig. VII-8 Pgs. 631-632, Sect. on *Channelized Three-Leg Intersections*	Pg. 2, 2nd col, Para. 1 Pg. 3. Para. 3 Pg. 18, Form 2 Pg. 21, Table 1	Pg. 1, Paras. 2-3 Pg. 21, Fig. 3-1 Pg. 24, Bottom fig. Pg. 25, Para. 3 Pg. 26, Top fig. Pg. 28, Middle fig. Pg. 32, Middle fig.	Pg. 319, Para. 4 Pgs. 384-385, Sect. on *Principles of Intersection Channelization*

3B.21, 3E.01, 3F.02, 3G.01 through 3G.06, & 5G.03	Pgs. 635-641, Sect. on *Channelized Four-Leg Intersections* Pgs. 674-689, Sects. on *Channelized Islands*, *Divisional Islands*, *Refuge Islands*, *Island Size and Designation*, *Delineation and Approach-End Treatment* & *Right-Angle Turns With Corner Island* Pgs. 740-749, Sects. on *General Design Considerations* & *Channelization* Pg. 778, Sect. on *Continuous Left-Turn Lanes (Two-Way)*	Pg. 22, Table 2 Pg. 26. 3nd col, Para. 2 Pg. 71, 5th bullet Pg. 99, Para. 3	Pg. 34, Para. 1 & bottom fig. Pg. 35, Bottom left fig. Pg. 38, Middle fig. Pg. 39, Paras. 2-3 & top two figs. Pg. 69, Sect. on *Traffic Islands* Pg. 74, Fig. 4-30 Pgs. 75-76, Para. 1 on 1st pg. & Sects. on *Guidelines for Design of Traffic Islands*, *Guidelines for Selection of Island Type*, & *Guidelines for Design of Median Islands* Pg. 79, Fig. 4-34 Pgs. 94- 95, Intersct. No. 4 Pgs. 102-103, Intersct. No. 8 Pgs. 106-113, Intersct. Nos. 10-13 Pgs. 116-117, Intersct. No. 15 Pgs. 132-133, Intersct. No. 22 Pgs. 138-139, Intersct. No. 29 Pgs. 148-153, Intersct. Nos. 35-37	Pg. 388, Sect. on *Traffic Island Design* Pg. 405, Para. 4 Pg. 434, Sect. on *Channelizing Lines* Pg. 435, Para. 4 Pg. 438, Item 5 Pg. 439, Para. 5 Pg. 440 Sect. on *Channelization* Pgs. 441-443, Sect. on *Channelizing Traffic Control Devices*

The spatial visual functions of acuity and contrast sensitivity are important in the ability to detect/recognize downstream geometric features such as pavement width transitions, channelized turning lanes, island and median features across the intersection, and any nonreflectorized raised elements at intersections. Visual acuity (the ability to see high-contrast, high-spatial-frequency stimuli, such as black letters on a white eye chart) shows a slow decline beginning at approximately age 40, and marked acceleration at age 60 (Richards, 1972). Approximately 10 percent of men and women between ages 65 and 75 have (best corrected) acuity worse than 20/30, compared with roughly 30 percent over the age of 75 (Kahn, Leibowitz, Ganley, Kini, Colton, Nickerson, and Dawber, 1977). A driver's response to intersection geometric features is influenced in part by the processing of high-spatial-frequency cues--for example, the characters on upstream advisory signs--but it is the larger, often diffuse edges defining lane and pavement boundaries, curb lines, and raised median barriers that are the targets with the highest priority of detection for safety. Older persons' sensitivity to visual contrast (the ability to see objects of various shapes and sizes under varying levels of contrast) also declines beginning around age 40, then declines steadily as age increases (Owsley, Sekuler, and Siemsen, 1983). Poor contrast sensitivity has been shown to relate to increased crash involvement for drivers age 66 and older, when incorporated into a battery of vision tests also including visual acuity and horizontal visual field size (Decina and Staplin, 1993).

The effectiveness of channelization from a safety perspective has been documented in several studies. An evaluation of Highway Safety Improvement Program projects showed that channelization produced an average benefit-cost ratio of 4.5 (FHWA, 1996). In this evaluation, roadway improvements consisting of turning lanes and traffic channelization resulted in a 47 percent reduction in fatal crashes, a 26 percent reduction in nonfatal injury crashes, and a 27 percent reduction in combined fatal plus nonfatal injury crashes, at locations where before and after exposure data were available.

One of the advantages of using curbed medians and intersection channelization is that it provides a better indication to motorists of the proper use of travel lanes at intersections. In a set of studies performed by the California Department of Public Works investigating the differences in crash experience with raised channelization versus channelization accomplished through the use of flush pavement markings, the findings were as follows: raised traffic islands are more effective than flush marked islands in reducing frequencies of night crashes, particularly in urban areas; and little difference is noted in the effectiveness of raised versus marked channelizing islands at rural intersections (Neuman, 1985).

One of the most common uses of channelization is for the separation of left-turning vehicles from the through-traffic stream. The safety benefits of left-turn channelization have been documented in several studies. A study by McFarland, Griffin, Rollins, Stockton, Phillips, and Dudek (1979) showed that crashes at signalized intersections where a left-turn lane was added, in combination with and without a left-turn signal phase, were reduced by 36 percent and 15 percent, respectively. At nonsignalized intersections with marked channelization separating the left-turn lane from the through lane, crashes were reduced for rural, suburban, and urban areas by 50, 30, and 15 percent, respectively. When raised channelization devices were used, the crash reductions were 60, 65, and 70 percent in rural, suburban, and urban areas, respectively. Consistent findings were reported in Hagenauer, Upchurch, Warren, and Rosenbaum (1982).

Important considerations in choosing to implement raised versus marked channelization include operating speed and type of maneuver (i.e., left turn versus right turn). Left-turn channelization separating through and turning lanes may, because of its placement, constitute a hazard when a raised treatment is applied, especially on high-speed facilities. Detection and avoidance of such hazards requires visual and response capabilities known to decline significantly with advancing age.

Another benefit in the use of channelization is the provision of a refuge for pedestrians. refuge islands are a design element that can aid older pedestrians who have slow walking speeds. With respect to the Hagenauer et al. (1982) study cited earlier, Hauer (1988) stated that because channelization in general serves to simplify an otherwise ambiguous and complex situation, the channelization of an existing intersection might enhance both the safety and mobility of older persons, as well as enhance the safety of other pedestrians and drivers. However, in designing a new intersection, he stated that the presence of islands is unlikely to offset the disadvantage of large intersection size for the pedestrian.

Staplin, Harkey, Lococo, and Tarawneh (1997) conducted a field study evaluating four right-turn lane geometries to examine the effect of channelized right-turn lanes and the presence of skew on right-turn maneuvers made by drivers of different ages. One hundred subjects divided across three age groups drove their own vehicles around test routes using the local street network in Arlington, VA. The three age groups were young/middle-aged (ages 25-45), young-old (ages 65-74), and old-old (age 75 and older). As diagrammed in figure 2, the four right-turn lane geometries were:

a) A nonchannelized 90-degree intersection where drivers had the chance to make a right turn on red (RTOR) around a 12.2-m (40-ft) radius. This site served as a control geometry to examine how channelized intersections compare with nonchannelized intersections.

(b) A channelized right-turn lane at a 90-degree intersection with an exclusive use (acceleration) lane on the receiving street. Under this geometric configuration, drivers did not need to stop at the intersection and they were removed from the conflicting traffic upon entering the cross street. They had the opportunity to accelerate in their own lane on the cross street and then change lanes downstream when they perceived that it was safe to do so.

(c) A channelized right-turn lane at a 65-degree skewed intersection without an exclusive use lane on the receiving street.

(d) A channelized right-turn lane at a 90-degree intersection without an exclusive use lane on the receiving street. Under this geometry, drivers needed to check the conflicting traffic and complete their turn into a through traffic lane on the cross street.

The right-turn maneuver at all locations was made against two lanes carrying through (conflicting) traffic. The two through lanes were the only ones that had a direct effect on the right-turn maneuver. All intersections were located on major or minor arterials within a growing urban area, where the posted speed limit was 56 km/h (35 mi/h). All intersections were controlled by traffic

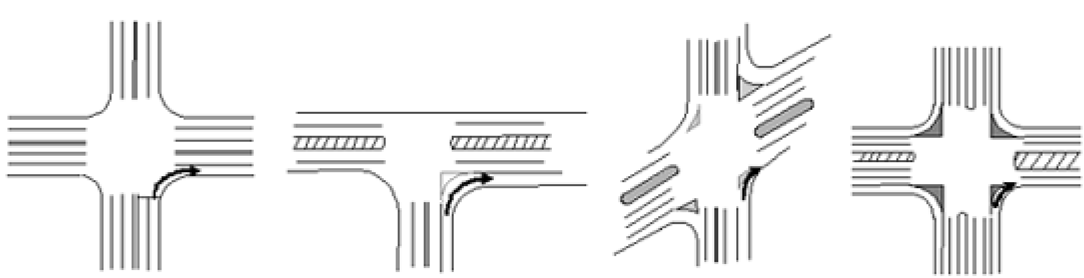


Figure 2. Intersection geometries examined in the Staplin et al.(1997) field study of right-turn channelization.

signals with yield control on the three channelized intersections.

The results indicated that right-turn channelization affects the speed at which drivers make right turns and the likelihood that they will stop before making a RTOR. Drivers, especially younger drivers (ages 25-45), turned right at speeds 4.8-8 km/h (3-5 mi/h) higher on intersection approaches with channelized right-turn lanes than they did on approaches with nonchannelized right-turn lanes.

At the nonchannelized intersection, 22 percent of the young/middle-aged drivers, 5 percent of the young-old drivers, and none of the old-old drivers performed a RTOR without a stop. On approaches with channelized right-turn lanes, young/middle-aged and young-old drivers were much less likely to stop before making a RTOR. Where an acceleration lane was available, 65 percent of the young/middle-aged drivers continued through without a complete stop, compared with 55 percent of the young-old drivers and 11 percent of the old-old drivers. Old-old female drivers *always* stopped before a RTOR. The increased mobility exhibited by the two younger groups of drivers at the channelized right-turn lane locations was not, however, exhibited by the old-old drivers (age 75 and older), who stopped in 19 of the 20 turns executed at the channelized locations. Also, questionnaire results indicated drivers perceived that making a right turn on an approach with a channelized right-turn lane *without an acceleration lane on the cross street* was more difficult than at other locations, and even more difficult than at skewed intersections.

Regarding channelization for mid-block left-turn treatments. Bonneson and McCoy (1997) evaluated the safety and operational effects of three mid-block left-turn treatments: raised curb medians; two-way, left turn-lanes; and undivided cross sections. Traffic flow data were collected during 32 field studies in 8 cities in 4 States, and 3-year crash histories for 189 street segments were obtained from cities in 2 States. The studies were conducted on urban or suburban arterial segments, and therefore recommendations can only be applied to such environments that include the following criteria: traffic volume exceeding 7,000 vehicles per day; speed limit between 48 and 80 km/h (30 and 50 mi/h); spacing of at least 107 m (350 ft) between signalized intersections; direct access from abutting properties; no angle curb parking (parallel parking is acceptable); located in or near a populated area (e.g., population of 20,000 or more); no more than six through lanes (three in each direction); and arterial length of at least 1.2 km (0.75 mi.).

In terms of annual delays to major-street left-turn and through vehicles, the raised-curb treatment has slightly higher delays than the TWLTL treatment at the highest left-turn and through volumes, which results from the greater likelihood of bay overflow for the raised-curb median treatment under high-volume conditions. The undivided cross section has significantly higher delays than the raised-curb treatment for all nonzero combinations of left-turn and through volume.

Looking at crash frequencies as a function of mid-block channelization treatment, the raised curb median treatment is associated with the fewest crashes of all three treatment types. Differences between the crash frequencies for TWLTL treatments vs undivided cross sections are affected by whether or not parallel parking is allowed on the undivided cross section. When parallel parking is allowed on the undivided cross section, the undivided cross section is associated with significantly more crashes than the TWLTL treatment. However, when parallel parking is not allowed, the TWLTL has about the same crash frequency as the undivided cross section at lower traffic volumes.

In general, at mid-block locations, the raised-curb median treatment was associated with fewer crashes than the undivided cross section and TWLTL, especially for average daily traffic demands greater than 20,000 vehicles per day. Also, a benefit of the raised-curb median is that it provides a pedestrian refuge.

Bonneson and McCoy (1997) provide a set of six tables to use as guidelines in considering the conversion of an undivided cross section to a raised curb median, or to a TWLTL, and conversion from a TWLTL to a raised-curb median treatment. In these tables, it is recommended that the existing treatment remain in place when the benefit-cost ratio (in terms of delay and safety) is less than 1.0, and when the benefit-cost ratio exceeds 2.0, it is recommended that the engineer consider adding the alternative treatment.

Bonneson and McCoy (1997) do not report crash frequencies by driver age, for one treatment versus another. However, approximately one-fifth of the older drivers participating in focus group studies conducted by Staplin, Harkey, Lococo, and Tarawneh (1997) reported that using center two-way left turn lanes (TWLTL), was confusing, risky, and made them uncomfortable, because at times they have come face-to-face with an opposing left turner, and both drivers were stranded. Also mentioned was the difficulty seeing the pavement markings in poor weather (night, fog, rain) when they are less visible, and particularly when they are snow covered. Drivers referred to TWLTL's as "suicide lanes." In the same research study, Staplin et al. (1997) reported on a crash analysis that revealed ways in which older drivers failed to use a TWLTL correctly: a TWLTL was not used for turning at all; and the TWLTL was entered too far in advance of where the turn was to be made.

D. Design Element: Intersection Sight Distance Requirements

Table 4. Cross-references of related entries for intersection sight distance requirements.

Applications in Standard Reference Manuals			
AASHTO Green Book (1994)	**Roadway Lighting Handbook (1978)**	**NCHRP 279 Intersection Channelization Design Guide (1985)**	**Traffic Engineering Handbook (1999)**
Pgs. 126-127, Sect. on *Decision Sight Distance* Pg. 440, Para. 5 Pg. 469, Para. 2 Pg. 491, Para. 1 Pg. 643, Para. 2 Pg. 645, Para. 1 Pgs. 646-647, Sect. on *Profile* Pgs. 696-724, Sects. on *Sight Distance* & *Stopping Sight Distance at Intersections for Turning Roadways* Pg. 796, Para. 5 through Pg. 801 Pgs. 938-939, Sects. on *Terminal Location and Sight Distance*, *Ramp Terminal Design*, & *Distance Between a Free-flow Terminal and Structure*	Pg. 18, Form 2 Pg. 22, Table 2 Pg. 25, Table 3 Example	Pg. 1, Item , 1st bullet Pg. 10, Table 2-4 Pgs. 13-14, Sect. on *Sight Distance* Pg. 15, Para. 1 Pg. 27, Bottom right fig. Pg. 30, 2nd fig. from bottom Pg. 31, Para. 3 Pg. 35, Para. 3 & bottom right fig. Pg. 44, Para. 6, item 1 Pg. 45, Table 4-2 Pg. 63, Para. 3, item 3 Pg. 75, Last item 4 Pgs. 99-103, Intersct. Nos. 6-8 Pgs. 106-111, Intersct. Nos. 10-12	Pg. 238, Sect. on *Intersection Sight Distance* Pg. 339, Para. 3 Pgs. 375-376, Sect. on *Inter-section Sight Distance (ISD)* Pg. 405, Para. 4

Because at-grade intersections define locations with the highest probability of conflict between vehicles, adequate sight distance is particularly important. Not surprisingly, a number of studies have shown that sight distance problems at intersections usually result in a higher crash rate (Mitchell, 1972; Hanna, Flynn, and Tyler, 1976; David and Norman, 1979). The need for adequate sight distance at an intersection is best illustrated by a quote from the Green Book: "The operator of a vehicle approaching an intersection at-grade should have an unobstructed view of the entire intersection and sufficient lengths of the intersecting highway to permit control of the vehicle to avoid collisions" (AASHTO, 1994). AASHTO values (for both uncontrolled and stop-controlled intersections) for available sight distance are measured from the driver's eye height (currently 1,070 mm [3.25 ft]) to the roofline of the conflicting vehicle (currently 1,300 mm [4.25 ft]).

Sight distances at an intersection can be reduced by a number of deficiencies, including physical obstructions too close to the intersection, severe grades, and poor horizontal alignment. The alignment and profile of an intersection have an impact on the sight distance available to the driver and thus affect the ability of the driver to perceive the actions taking place both at the intersection and on its approaches. Since proper perception is the first key to performing a safe maneuver at an intersection, it follows that sight distance should be maximized; this, in turn, means that the horizontal alignment should be straight and the gradients as flat as practical. Horizontal curvature on the approaches to an intersection makes it difficult for drivers to determine appropriate travel paths, because their visual focus is directed along lines tangential to these paths. Kihlberg and Tharp (1968) showed that crash rates increased 35 percent for highway segments with curved intersections over highway segments with straight intersections. Limits for vertical alignment at intersections suggested by AASHTO (1994) and Institute of Transportation Engineers (1984) are 3 and 2 percent, respectively.

Harwood, Mason, Pietrucha, Brydia, Hostetter, and Gittings (1993) stated that the provision of intersection sight distance (ISD) is intended to give drivers an opportunity to obtain the information they need to make decisions about whether to proceed, slow, or stop in situations where potentially conflicting vehicles may be present. They noted that while it is desirable to provide a reasonable margin of safety to accommodate incorrect or delayed driver decisions, there are substantial costs associated with providing sight distances at intersections; therefore, it is important to understand the derivation of ISD requirements and why it is reasonable to expect a safety benefit from tailoring this design parameter to the needs of older drivers.

Traditionally, the need for--as well as the basis for calculating--sight distances at intersections has rested upon the notion of the *sight triangle*. This is diagrammed in figure 3. As excerpted from NCHRP Report 383, this diagram effectively illustrates how different driver decisions during a (minor) road approach to an intersection (with a major road) depend upon the planned action. The driver's first decision is to either stop or to continue through the intersection (with a turning or a crossing maneuver) according to the type of traffic control information he or she perceives. A red signal or a stop sign results in a "stop" decision; all other types of information are functionally equivalent at this stage of driver decision making, translating into a "yield" decision. That is, drivers' decisions at this stage are dichotomous: (1) slow down and prepare to stop, regardless of traffic on the major road, or (2) based on their view of the major road, either slow down, maintain speed, or accelerate as required to safely complete their intended maneuver. For drivers who are required to stop, their decision to proceed after the stop also is based

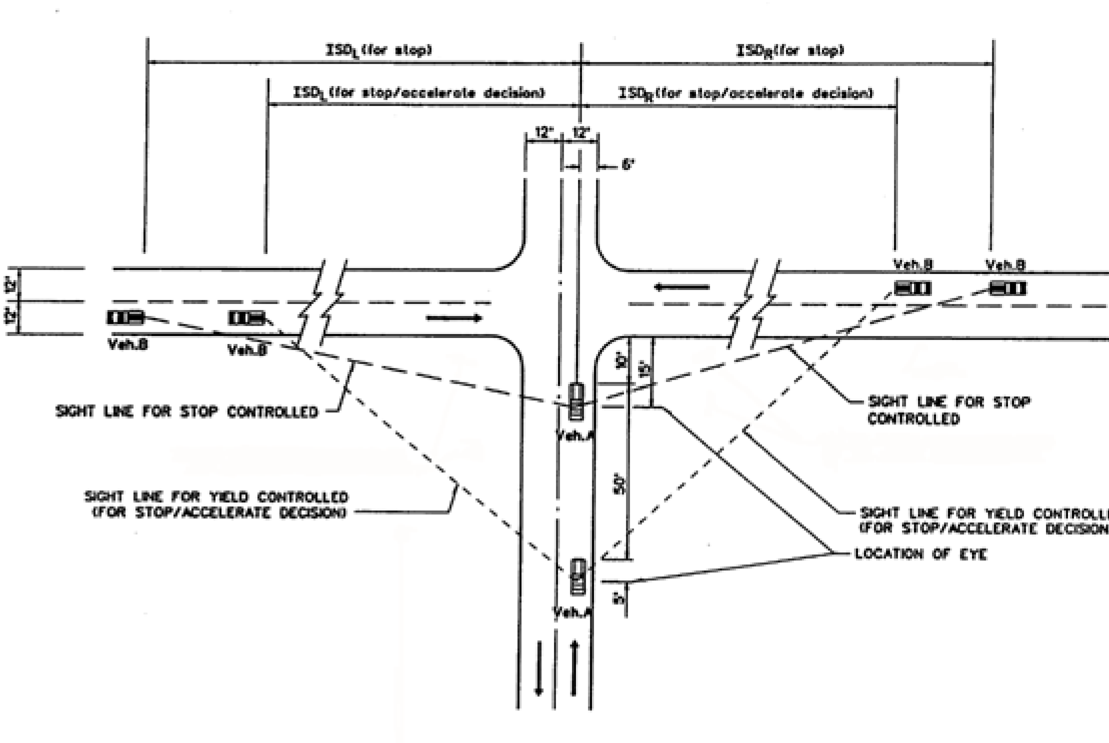


Figure 3. Sight distance for left and right turns for passenger car drivers at yield-controlled intersections. Source: Harwood et al. (1993).

on a view of traffic on the major road, but at a point much closer to the intersection. The contrasting sight lines and sight triangles defined by the position of a driver who *must* stop before proceeding at the intersection, versus one who *may* proceed without stopping, conditional on the intersecting (major) road traffic, are clearly indicated in figure 3.

For purposes of describing driver decision making, the diagram in figure 3 may apply to varying aspects of intersection operations in all Cases I through IV as *per* current AASHTO classification. For Case V, however, where a driver is turning left from a major road at an intersection or driveway, the decision process and corresponding sight distance requirements are defined differently. The sight lines in this case are defined by the presence, type (passenger versus heavy vehicle), and location (positioned or unpositioned in the intersection) of opposing left-turning traffic, and by the lateral offset of the opposite left-turn lanes themselves. These relationships are illustrated in figure 4 from McCoy, Navarro, and Witt (1992).

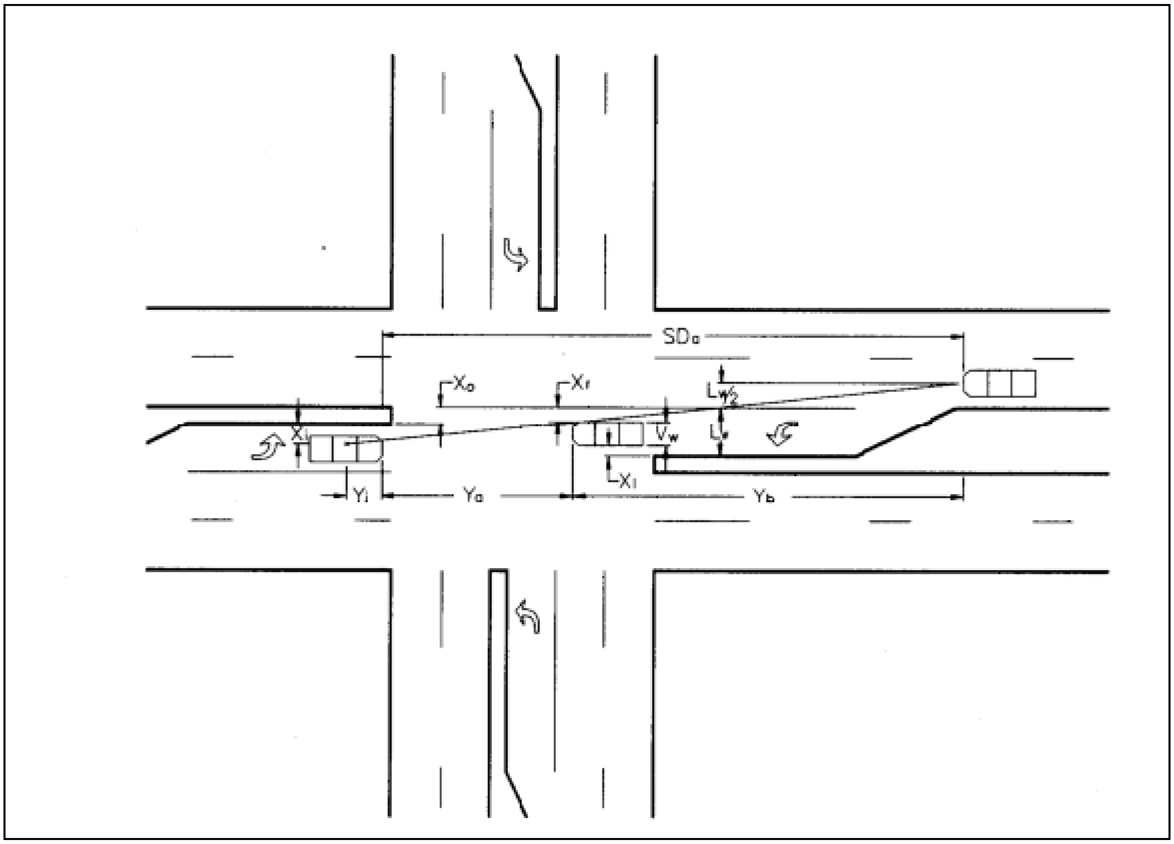


Figure 4. Spatial relationships that determine available sight distance.
Source: McCoy, Navarro, and Witt (1992).

It is also important to acknowledge recent thinking (*cf.* Harwood et al. 1993) that bases recommended sight distances upon the observed gaps that drivers will accept for performing various maneuvers at intersections--specifically, upon the "critical gap." This is the distance, expressed in the number of seconds (at operating speed) of separation between a subject vehicle and a conflict vehicle, where the driver of the subject vehicle will make a decision to proceed with a maneuver *ahead of* the conflict vehicle 50 percent of the time. The "Gap Acceptance" model yields different, typically shorter ISD requirements, than the existing (or modified) AASHTO model. It also classifies intersection operations in a somewhat different manner than AASHTO (1994).

Both the assumptions underlying the current AASHTO (1994) model and the Gap Acceptance model, respectively, received consideration in developing the Handbook's recommendations for this design element. Apart from the theoretical differences between these models, there is also the practical matter that it may take some time for designers and engineers who are familiar with and have worked successfully with one approach to embrace an alternative approach. Thus, this Handbook seeks to accommodate the full range of design practices, to the extent that data provide an understanding of the intersection sight distance requirements of older drivers. Where ISD requirements are defined through application of formulas incorporating "perception-reaction time" (PRT), the broad and well-documented age differences in this aspect of driver performance support recommendations for all included cases (I through V). Where ISD requirements are determined through a formula that depends upon gap size, however, recommendations must be limited at this time to cases where gap acceptance by older versus younger drivers has been empirically studied (Case F).

The rationale for recommendations pertaining to intersection sight distance requirements will proceed as follows. First, driver age differences in cognitive and physical capabilities that are relevant to ISD issues will be discussed. Then, research efforts that have attempted to quantify the safety impact of providing adequate sight distance are summarized, plus studies examining the appropriate values for specific components used when calculating sight distance in the AASHTO and Gap Acceptance models.

Older road users do not necessarily react more slowly to events that are expected, but they take significantly longer to make decisions about the appropriate response than younger road users, and this difference becomes more exaggerated in complex situations. Although the cognitive aspects of safe intersection negotiation depend upon a host of specific functional capabilities, the net result is response slowing. There is general consensus among investigators that older adults tend to process information more slowly than younger adults, and that this slowing not only transcends the slower reaction times often observed in older adults but may, in part, explain them (Anders, Fozard, and Lillyquist, 1972; Eriksen, Hamlin, and Daye, 1973; Waugh, Thomas, and Fozard, 1978; Salthouse and Somberg, 1982; Byrd, 1984). Of course, a conflict must be *seen* before any cognitive processing of this sort proceeds. Therefore, any decrease in available response time because of sight distance restrictions will pose disproportionate risks to older drivers. Slower reaction times for older versus younger adults when response uncertainty is increased has been demonstrated by Simon and Pouraghabagher (1978), indicating a disproportionately heightened degree of risk when older road users are faced with two or more choices of action. Also, research has shown that older persons have greater difficulty in situations where planned actions must be rapidly altered (Stelmach, Goggin, and Amrhein, 1988). The difficulty older persons experience in making extensive and repeated head movements further increases the decision and response times of older drivers at intersections.

Table 5. Expected reduction in number of crashes per intersection per year. Source: David and Norman, 1979.

AADT* (1000s)	Increased Sight Distance (ft)		
	20-49	**50-99**	**>100**
< 5	0.18	0.20	0.30
5 - 10	1.00	1.30	1.40
10 - 15	0.87	2.26	3.46
> 15	5.25	7.41	11.26

** annual average daily traffic entering the intersection*

David and Norman (1979) quantified the relationship between available sight distance and the expected reduction in crashes at intersections. The results of this study showed that intersections with shorter sight distances generally have higher crash rates. Using these results, predicted crash reduction frequencies related to ISD were derived as shown in table 5.

Other studies have attempted to show the benefits to be gained from improvements to ISD (Mitchell, 1972; Strate, 1980). Mitchell conducted a before-and-after analysis, with a period of 1 year on each end, of intersections where a variety of improvements were implemented. The results showed a 67 percent reduction (from 39 to 13) in crashes where obstructions that inhibited sight distance were removed; this was the most effective of the implemented improvements. Strate's analysis examined 34 types of improvements made in Federal Highway Safety Program projects. The results indicated that sight distance improvements were the most cost-effective, producing a benefit-cost ratio of 5.33:1. The more recent report on the FHWA Highway Safety Improvement Programs (1996) indicates that improvements in intersection sight distance have a benefit-cost ratio of 6.1 in reducing fatal and injury crashes. In these analyses, fatal crashes were reduced by 56 percent and nonfatal injury crashes by 37 percent after sight distance improvements were implemented.

Collectively, the studies described above indicate a positive relationship between available ISD and a reduction in crashes, though the amount of crash reduction that can be expected by a given increase in sight distance may be expected to vary according to the maneuver scenario and existing traffic control at the intersection. Procedures for determining appropriate ISD's are provided by AASHTO for various levels of intersection control and the maneuvers to be performed. The scenarios defined are as follows:

- Case I: No Control. ISD for vehicles approaching intersections with no control, at which vehicles are not required to stop, but may be required to adjust speed.
- Case II: Yield Control. ISD for vehicles on a minor-road approach controlled by a yield sign.
- Case IIIA: Stop Control--Crossing Maneuver. ISD for a vehicle on a stop-controlled approach on the minor road to accelerate from a stopped position and cross the major road.
- Case IIIB: Stop Control--Left Turn. ISD for a vehicle on a stop-controlled approach on the minor road to accelerate from a stopped position and turn left onto the major road.

- Case IIIC: Stop Control--Right Turn. ISD for a vehicle on a stop-controlled approach on the minor road to accelerate from a stopped position and turn right onto the major road.
- Case IV: Signal Control (should be designed by Case III conditions). ISD for a vehicle on a signal-controlled approach.
- Case V: Stop Control--Vehicle Turning Left From Major Highway Into a Minor Roadway. ISD for a vehicle stopped on a major road, waiting to turn left across opposing lanes of travel.

By comparison, for applications of the Gap Acceptance model, an alternative classification system has been proposed (Harwood et al., 1996):

- Case A: Intersections with no control.
- Case B: Intersections with Stop control on the minor road.

  Case B1: Left turn from the minor road.
  Case B2: Right turn from the minor road.
  Case B3: Crossing maneuver from the minor road

- Case C: Intersections with Yield control on the minor road.

  Case C1: Crossing maneuver from the minor road.
  Case C2: Left or right turn from the minor road.

- Case D: Intersections with traffic signal control.
- Case E: Intersections with all-way Stop control.
- Case F: Intersections where a stopped vehicle is turning left from a major road.

One of the principal components in determining ISD in all cases defined according to AASHTO (1994) is perception-reaction time (PRT). The discussion of this value is first presented in chapters 2 and 3 of the Green Book under "Reaction Time" and "Brake Reaction Time," respectively (AASHTO, 1994). Results of several studies (e.g., Normann, 1953; Johansson and Rumar, 1971) are cited, and in conclusion, the 2.5-s value is selected since it was found to be adequate for approximately 90 percent of the overall driver population. Controlled field studies and simulator studies involving older drivers have confirmed that brake reaction times to unexpected hazards (e.g., a barrel rolling into the road in front of the driver; a vehicle turning in front of a driver who is traveling straight through an intersection) are not significantly different as a function of age, and that virtually all response times are captured by the current 2.5-s AASHTO design parameter for brake perception-response time (Lerner, Huey, McGee, and Sullivan, 1995; Kloeppel, Peters, James, Fox, and Alicandri, 1995).

With respect to at-grade intersections, AASHTO recommends the following values of PRT for ISD calculations. In Case I, the PRT is assumed to be 2.0 s plus an additional 1.0 s to actuate braking, although the "preferred design" uses stopping sight distance (SSD) as the ISD design value (which incorporates a PRT of 2.5 s). In Case II, SSD is the design value; thus, the PRT is 2.5 s. For all Case III scenarios and Cases IV and V, the PRT is assumed to be 2.0 s.

Regarding PRT for Cases III and V, the value of 2.0 s assumed by AASHTO (1994) represents the time necessary for the driver to look in both directions of the roadway, to perceive that there is sufficient time to perform the maneuver safely, and to shift gears, if necessary, prior to starting. This value is based on research performed by Johansson and Rumar (1971). The PRT is defined as the time from the driver's first look for possible oncoming traffic to the instant the car begins to move. Some of these operations are done simultaneously by many drivers, and some operations, such as shifting gears, may be done before searching for intersecting traffic or may not be required with automatic transmissions. AASHTO states that a value of 2.0 s is assumed to represent the time taken by the slower driver.

A critique of these values questioned the basis for reducing the PRT from 2.5 s used in SSD calculations to 2.0 s in the Case III ISD calculations (Alexander, 1989). As noted by the author, "The elements of PRT are: detection, recognition, decision, and action initiation." For SSD, this is the time from object or hazard detection to initiation of the braking maneuver. Time to search for a hazard or object is not included in the SSD computation, and the corresponding PRT value is 2.5 s. Yet, in all Case III scenarios, the PRT has been reduced to 2.0 s and now includes a search component which was not included in the SSD computations. Alexander pointed out that a driver is looking straight ahead when deciding to perform a stopping maneuver and only has to consider what is in his/her forward view. At an intersection, however, the driver must look forward, to the right, and to the left. This obviously takes time, especially for those drivers with lower levels of physical dexterity, e.g., older drivers. Alexander (1989) proposed the addition of a "search time" variable to the current equations for determining ISD, and use of the PRT value currently employed in the SSD computations (i.e., 2.5 s) for all ISD computations. Neuman (1989) also argued that a PRT of 2.5 s for SSD may not be sufficient in all situations, and can vary from 1.5 s to 5.0 s depending on the physical state of the driver (alert versus fatigued), the complexity of the driving task, and the location and functional class of the highway.

A number of research efforts have been conducted to determine appropriate PRT values for use in ISD computations. Hostetter, McGee, Crowley, Seguin, and Dauber (1986) examined the PRT of 124 subjects traversing a 3-hour test circuit which contained scenarios identified above as Cases II, IIIA, IIIB, and IIIC. For the Case II (yield control) scenario, the results showed that in over 90 percent of the trials, subjects reacted in time to meet the SSD criteria established and thus the 2.5-s PRT value was adequate. With respect to Case III scenarios, the PRT was measured from the first head movement after a stop to the application of the accelerator to enter the intersection. The mean and 85th percentile values for all maneuvers combined were 1.82 s and 2.7 s, respectively. The results also showed that the through movement produced a lower value than the mean, while the turning maneuvers produced a higher value. These results lead to conclusions that the 2.0-s criteria for Case IIIA be retained and that the PRT value for the Case III turning maneuvers (B and C) be increased from 2.0 to 2.5 s. One other result, which is applicable to the current effort, was that no significant differences were found with respect to age, i.e., increased PRTs were needed to accommodate all drivers.

Fambro, Koppa, Picha, and Fitzpatrick (1998) found significant differences in mean perception-brake response times as a function of age and gender, with older drivers and female drivers demonstrating longer response times. They conducted three separate on-road studies to measure driver perception-brake response time to several stopping sight distance situations. Studies were conducted on a closed course as well as on an open roadway. In one study conducted on the closed course, subjects drove an instrumented test vehicle belonging to the Texas Transportation Institute, and in another closed course study they drove their own vehicles. In the open roadway study, they drove their own vehicles. Seventeen younger drivers (age 24 or under) and 21 older drivers (age 55 or older) participated in trials that required them to brake in response to expected and unexpected events, that included a barrel rolling off of a pick up truck parked next to the roadway, an illuminated LED on the windshield, and a horizontal blockade that deployed ahead of them on the roadway. Across all *expected*-object, perception-brake response time trials, the mean response time for younger drivers was 0.52 s and the mean response time for older drivers was 0.66 s. For these "expected" trials, the mean perception brake-response time for males was 0.59 s and for females was 0.63 s. For the *unexpected*-object, perception-brake response trials, longer response times were demonstrated for trials where subjects drove their own vehicles, compared to those in which they drove the Transportation Institute's vehicle. The study authors suggested that subjects were more relaxed and unsuspecting when driving their own vehicles. The mean response time across studies (controlled and open road, own vehicle and research vehicle) for the unexpected object was 1.1 s; the 95th percentile perception-brake response time was 2.0 s.

Based on this finding, Fambro et al. (1998) concluded that AASHTO's 2.5-s perception-brake reaction time value is appropriate for highway design, when stopping sight distance is the relevant control. However, they note that at locations or for geometric features where something other than stopping sight distance is the relevant control, different perception-reaction times may be appropriate. For example, longer perception-reaction times may be appropriate for intersection or interchange design where more complex decisions and driver speed and/or path correction are required.

Another effort examined the appropriateness of the PRT values currently specified by AASHTO for computing SSD, vehicle clearance interval, sight distance on horizontal curves, and ISD (McGee and Hooper, 1983). With respect to ISD, the results showed the following: for Case I, the driver is not provided with sufficient time or distance to take evasive action if an opposing vehicle is encountered; and for Case II, adequate sight distance to stop before arriving at the intersection is not provided despite the intent of the standard to enable such action. With respect to the PRT values, recommendations include increasing the 2.0-s and 2.5-s values used in Case I and Case II calculations, respectively, to 3.4 s. It was also recommended that the PRT value for Case III scenarios be redefined.

Although there is no consensus from the above studies on the actual values of PRT that should be employed in the ISD computations, there is a very clear concern as to whether the current values are meeting the needs of older drivers. Since older drivers tend to take longer in making a decision, especially in complex situations, the need to further evaluate current PRT values is underscored. Slowed visual scanning of traffic on the intersecting roadway by older drivers has been cited as a cause of near misses of (crossing) crashes at intersections during on-road evaluations. In the practice of coming to a stop, followed by a look to the left, then to the right, and then back to the left again, the older driver's slowed scanning behavior allows approaching vehicles to have closed the gap by the time a crossing maneuver finally is initiated. The traffic situation has changed when the older driver actually begins the maneuver, and drivers on the main roadway are often forced to adjust their speed to avoid a collision. Hauer (1988) stated that "the standards and design procedures for intersection sight triangles should be modified because there is reason to believe that when a passenger car is taken as the design vehicle, the sight distance is too short for many older drivers, who take longer to make decisions, move their heads more slowly, and wish to wait for longer gaps in traffic."

In contrast, recent research conducted by Lerner, Huey, McGee, and Sullivan (1995) concluded that, based on older driver performance, no changes to design PRT values were recommended for ISD, SSD, or decision sight distance (DSD), even though the 85th percentile J values exceeded the AASHTO 2.0-s design standard at 7 of the 14 sites. The J value equals the sum of the PRT time and the time to set the vehicle in motion, in seconds. No change was recommended because the experimental design represented a worst-case scenario for visual search and detection (drivers were required to begin their search only after they had stopped at the intersection and looked inside the vehicle to perform a secondary task). Naylor and Graham (1997), in a field study of older and younger drivers waiting to turn left at stop-controlled intersections (Case IIIB), similarly concluded that the current AASHTO value of 2.0 s is adequate for the PRT (J-value) used in calculating intersection sight distance at these sites.

Lerner et al. (1995) conducted an on-road experiment to investigate whether the assumed values for Case III driver PRT used in AASHTO design equations adequately represent the range of actual PRT for older drivers. Approximately 33 subjects in each of three driver age groups were studied: ages 20-40, ages 65-69, and age 70 and older. Drivers operated their own vehicles on actual roadways, were not informed that their response times were being measured, and were naive as to the purpose of the study (i.e., they were advised that the purpose of the experiment was to judge road quality and how this relates to aspects of driving). The study included 14 data collection sites on a 90-km (56-mi) long route. Results showed that the older drivers did *not* have longer PRT than younger drivers, and in fact the 85th percentile PRT closely matched the AASHTO design equation value of 2.0 s. The 90th percentile PRT was 2.3 s, with outlying values of 3 to 4 s. The median daytime PRT was approximately 1.3 s. Interestingly, it was found that typical driver actions did not follow the stop/search/decide maneuver sequence implied by the model; in fact, drivers continued to search and appeared ready to terminate or modify their maneuver even after they had begun to move into the intersection. This finding resulted in the study authors' conclusion that the behavioral model on which ISD is based is conservative.

Harwood, Mason, Brydia, Pietrucha, and Gittings (1996) evaluated current AASHTO policy on ISD for Cases I, II, III, IV, and V during performance of NCHRP project 15-14(1), based on a survey of current highway agencies' practices and a consideration of alternative ISD models and computational methodologies, as well as findings from observational studies for selected cases. Although this work culminated in recommendations for minimum distances for the major and minor legs of the sight triangle for all cases, driver age was not included as a study variable; therefore, specific values for these design elements were not included within the recommendations presented in this Handbook, nor is an exhaustive discussion of these materials included in this section. The results of the Harwood et al. (1996) analyses pertaining to ISD for Case IIIB and IIIC--and by extension for Case V--are of particular interest, however, in the interpretation of other, related findings from an older driver field study in this area. These analysis outcomes are reviewed below.

Prior to the 1990 AASHTO Green Book, the issue of ISD for a driver turning left off of a major roadway onto a minor roadway or into an entrance (Case V) was not specifically addressed. In the 1990 Green Book, the issue was addressed at the end of the Case III discussions in two paragraphs. In the 1994 Green Book, these same paragraphs have been placed under a new condition referred to as Case V. The equation used for determining ISD for Case V was simply taken from the Case IIIA (crossing maneuver at a stop-controlled intersection) and Case IIIB (left-turn maneuver from a stop-controlled minor road onto a major road) conditions, with the primary difference between the cases being the distance traveled during the maneuver. A central issue in defining the ISD for Case V involves a determination of whether the tasks that define ISD for Cases IIIA and IIIB are similar enough to the tasks associated with Case V to justify using the same equation, which follows:

$$ISD=1.47\ V\ (J+t_a)\ \text{English}$$

$$ISD=0.278\ V\ (J+t_a)\ \text{Metric}$$

where:

ISD = intersection sight distance (feet for English equation; meters for metric equation).

V = major roadway operating speed (mi/h for English equation; km/h for metric equation).

J = time required to search for oncoming vehicles, to perceive that there is sufficient time to make the left turn, and to shift gears, if necessary, prior to starting (J is currently assumed to be 2.0 s).

t_a= time required to accelerate and traverse the distance to clear traffic in the approaching lane(s); obtained from figure IX-33 in the AASHTO Green Book.

For Case IIIA (crossing maneuver), the sight distance is calculated based on the need to clear traffic on the intersecting roadway on both the left and right sides of the crossing vehicle. For Case IIIB (left turn from a stop), sight distance is based on the requirement to first clear traffic approaching from the left and then enter the traffic stream of vehicles from the right. It has been demonstrated that the perceptual judgments required of drivers in both of these maneuver situations increase in difficulty when opposing through traffic must be considered.

The perceptual task of turning left from a major roadway at an unsignalized intersection or during a permitted signal phase at a signalized intersection requires a driver to make time-distance estimates of a longitudinally moving target as opposed to a laterally moving target. Lateral movement (also referred to as tangential movement) describes a vehicle that is crossing an observer's line of sight, moving against a changing visual background where it passes in front of one fixed reference point after another. Longitudinal movement, or movement in depth, results when the vehicle is either coming toward or going away from the observer. In this case there is no change in visual direction, only subtle changes in the angular size of the visual image, typically viewed against a constant background. Longitudinal movement is a greater problem for drivers because the same displacement of a vehicle has a smaller visual effect than when it moves laterally--that is, lateral movement results in a much higher degree of relative motion (Hills, 1980).

In comparison with younger subjects, a significant decline for older subjects has been reported in angular motion sensitivity. In a study evaluating the simulated change in the separation of taillights indicating the overtaking of a vehicle, Lee (1976) found a threshold elevation greater than 100 percent for drivers ages 70-75 compared with drivers ages 20-29 for brief exposures at night. Older persons may in fact require twice the rate of movement to perceive that an object is approaching, versus maintaining a constant separation or receding, given a brief duration (2.0 s) of exposure. In related experiments, Hills (1975) found that older drivers required significantly longer to perceive that a vehicle was moving closer at constant speed: at 31 km/h (19 mi/h), decision times increased 0.5 s between ages 20 and 75. This body of evidence suggests that the 2.0-s PRT (i.e., variable J in the ISD equation above) used for Cases III and V may not be sufficient for the task of judging gaps in opposing through traffic by older drivers. A revision of Case V to determine a minimum required sight distance value which more accurately reflects the perceptual requirements of the left-turn task may therefore be appropriate.

Harwood et al. (1996) suggested that at locations where left turns from the major road are permitted at intersections and driveways, at unsignalized intersections, and at signalized intersections without a protected turn phase, sight distance along the major road should be provided based on a critical gap approach, as was recommended for left and right turns from the minor road at stop-controlled intersections. The Gap Acceptance model developed and proposed to replace the current ISD AASHTO model is:

$$ISD = 1.47\ VG \quad \text{English}$$

$$ISD = 0.278\ VG \quad \text{Metric}$$

where:

ISD = intersection sight distance (feet for English equation; meters for metric equation).

V = operating speed on the major road (mi/h for English equation, km/h for metric equation).

G = the specified critical gap (in seconds); equal to 5.5 s for crossing one opposing lane plus an additional 0.5 s for each additional opposing lane.

Field data were collected in the NCHRP study to better quantify the gap acceptance behavior of passenger car and truck drivers, but only for left- and right-turning maneuvers from minor roadways controlled by a STOP sign (Cases IIIB and C). In the Phase I interim report produced during the conduct of the NCHRP project, Harwood et al. (1993) reported that the critical gap currently used by the California Department of Transportation is 7.5 s. When current AASHTO Case IIIB ISD criteria are translated to time gaps in the major road traffic stream, the gaps range from 7.5 s (67 m [220 ft]) at a 32-km/h (20-mi/h) operating speed to 15.2 s (475 m [1,560 ft]) at a 112-km/h (70-mi/h) operating speed. Harwood et al. (1993) stated that the rationale for gap acceptance as an ISD criterion is that drivers safely accept gaps much shorter than 15.2 s routinely, even on higher speed roadways.

In developing the Gap Acceptance model for Case V, Harwood et al. (1996) relied on data from studies conducted by Kyte (1995) and Micsky (1993). Kyte (1995) recommended a critical gap value of 4.2 s for left turns from the major road by passenger cars for inclusion in the unsignalized intersection analysis procedures presented in the *Highway Capacity Manual* (Transportation Research Board, 1994). A constant value was recommended regardless of the number of lanes to be crossed; however, a heavy-vehicle adjustment of 1.0 s for two-lane highways and 2.0 s for multilane highways was recommended. Harwood et al. (1996) reported that Micsky's 1993 evaluation of gap acceptance behavior for left turns from the major roadway at two Pennsylvania intersections resulted in critical gaps with a 50 percent probability of acceptance (determined from logistic regression) of 4.6 s and 5.3 s. Using the rationale that design policies should be more conservative than operational criteria such as the *Highway Capacity Manual*, Harwood et al. (1996) recommended a critical gap for left turns from the major roadway of 5.5 s, and an increase in the critical gap to 6.5 s for left turns by single-unit trucks and to 7.5 s for left turns by combination trucks. In addition, if the number of opposing lanes to be crossed exceeds one, an additional 0.5 s per additional lane for passenger cars and 0.7 s per additional lane for trucks was recommended.

It is important to note that the NCHRP study did not consider driver age as a variable. However, Lerner et al. (1995) collected judgments about the acceptability of gapsin traffic as a function of driver age for left turn, right turn, and through movements at stop-controlled intersections. While noting that these authors found no significant differences between age groups in the *total* time required to perceive, react, and complete a maneuver in a related Case III PRT study, the Lerner et al. (1995) findings indicate that younger drivers accept shorter gaps than older drivers. The 50th percentile gap acceptance point was about 7 s (i.e., if a gap is 7 s long, only about half of the subjects would accept it). The 85th percentile point was approximately 11 s. The oldest group required about 1.1 s longer than the youngest group.

Staplin, Harkey, Lococo, and Tarawneh (1997) conducted an observational field study of driver performance as a function of left-turn lane geometry and driver age, at four locations where the main road operating speed was 56 km/h (35 mi/h). The mean left-turn critical gap sizes (in seconds) across all sites, for drivers who had positioned their vehicles within the intersection, were as follows: 5.90 s for the young/middle-aged (ages 25-45) females; 5.91 s for the young/middle-aged males; 6.01 s for the young-old (ages 65-74) females; 5.84 s for the young-old males; 6.71 s for the old-old (age 75 and older) females; and 6.55 s for the old-old males. Prominent trends indicated that older drivers demonstrated larger critical gap values at all locations. The young/middle-aged and young-old groups were not significantly different from each other; however, both were significantly different from the old-old group. Critical gap data were not collected in this study for drivers who did not position themselves within the intersection, but it is important to note that the older drivers were less likely to position themselves within the intersection than the young and middle-aged drivers.

Critical gap sizes displayed in a laboratory simulation study in the same project, where oncoming vehicles traveling at 56 km/h (35 mi/h) were viewed on a large screen display in correct perspective, ranged from 6.4 s to 8.1 s for young/middle-aged drivers and from 5.8 to 10.0 s for drivers age 75 and older. This increase in size and variability of the critical gap for left turns by older drivers suggests that the value for G in the Gap Acceptance model must be increased to accommodate this user group, beyond levels recommended in NCHRP 383 (where the performance of older drivers, *per se*, was not at issue).

The culmination of this work was a rigorous exercise of competing models and theoretical approaches for calculating sight distance requirements. As reported by Staplin et al. (1997) current and proposed sight distance models were exercised using data collected in the observational field study. This study was conducted at four intersections which differed in the amount that the opposite left-turn lanes were offset The goal was to determine which model(s), including existing and modified AASHTO models and a Gap Acceptance model, best predicted the data observed in the field.

Several data elements collected in the field received special attention. One of these data elements was the maneuver time of the left-turning driver. This time is equivalent to t_a in the AASHTO model, reference figure IX-33 in the AASHTO (1994) Green Book. These times were measured at each of four intersections included in the study, for positioned and unpositioned drivers. That is, separate maneuver-time measures were obtained, depending on whether or not the drivers positioned themselves within the intersection prior to turning. Staplin et al. (1997) found no significant differences in maneuver time as a function of age for the drivers turning left at the four intersections studied (which had distances ranging from 26 to 32 m [84 to 106 ft]). Maneuver times for drivers positioned within the intersection versus unpositioned drivers, however, were significantly different. Since older drivers less frequently positioned themselves in the field study, the design value for this factor (maneuver time) should be based on that obtained for unpositioned drivers.

A comparison between AASHTO values and the 95th percentile clearance times demonstrated by positioned drivers and unpositioned drivers in this study is presented in table 6. In table 6, the "positioned" vehicles were located within the intersection, approaching the median or centerline of the cross street. The "unpositioned" vehicles were at or behind the stop line or end of the left-turn bay. (See figure 8 located in the discussion for Design Element E, for an illustration of driver positioning within an intersection).

Table 6. Comparison of clearance times obtained in the Staplin et al. (1997) field study with AASHTO Green Book values used in sight distance calculations.

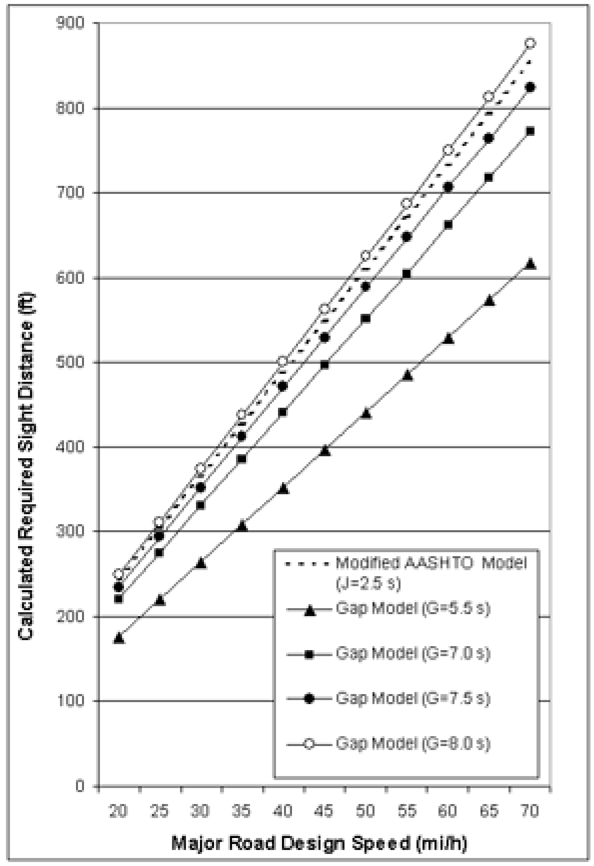
Measure	Vehicle Location	Left-Turn Lane Geometry			
		-4.3 m (-14 ft) Offset	-1 m (-3 ft) Offset	0 m (0 ft) Offset	+1.8 m (+ 6 ft) Offset
Distance Traveled (ft)	Positioned	21.3 m (70 ft)	20.4 m (67 ft)	19.5 m (64 ft)	21.3 m (70 ft)
95th Percentile Clearance Time (s) From Field Study	Positioned	3.8 s	3.9 s	3.9 s	3.9 s
AASHTO Clearance Time (s) From Figure IX-33	Positioned	5.1 s	5.0 s	5.0 s	5.1 s
Distance Traveled (ft)	Unpositioned	32.3 m (106 ft)	29.9 m (98 ft)	25.6 m (84 ft)	26.8 m (88 ft)
95th Percentile Clearance Time (s) From Field Study	Unpositioned	6.7 s	6.4 s	6.6 s	5.7 s
AASHTO Clearance Time (s) From Figure IX-33	Unpositioned	6.5 s	6.2 s	5.9 s	6.0 s

A detailed discussion of the outputs from the model exercise is provided in the publication *Intersection Geometric Design and Operational Guidelines for Older Drivers and Pedestrians* (Staplin, Harkey, Lococo, and Tarawneh, 1997). However, the most significant result for purposes of this discussion is as follows: *the required sight distances computed using a modified AASHTO model (where PRT was increased to 2.5 s) produced values that were most predictive of actual field operations.*

Thus, when ISD is calculated using the AASHTO model as it relates to drivers turning left from a major roadway, there is evidence that the PRT value should be increased to 2.5 s to provide adequate sight distance. The Gap Acceptance model, on the other hand, produced sight distance values that were approximately 23 percent shorter than the current AASHTO model, that uses a PRT of only 2.0 s. If the Gap Acceptance model is going to be used, particularly where there are significant volumes of older left-turning drivers, an adjustment factor applied to increase the sight distance to better accommodate this driver age group therefore appears warranted.

To determine what adjustment is most appropriate in this regard, a set of analyses were performed in which the goal was to identify a value of G that would yield required sight distance values meeting or exceeding those derived from the modified AASHTO model formula (i.e., where J = 2.5 s). By extension, this result would also best match the behavior of drivers 75 and older observed in the field study.

Very simply, alternate values for G were substituted into the gap formula for calculating minimum required sight distance (1.47VG). These included 5.5 s, as recommended by Harwood et al.(1996), as well values which increase in 0.5 s increments. The results of these calculations for alternate values of G, beginning at 7.0 s, are plotted against the required sight distance calculated using the modified AASHTO formula [$1.47V(J+t_a)$; where J=2.5 s and t_a is obtained from table IX-33 in the Green Book] in figure 5. As shown in this figure, a gap of 8.0 s affords sight distance for left-turning drivers that equals or exceeds the requirements calculated using the modified AASHTO model for major road design speeds from 32 km/h to 113 km/h (20 mi/h to 70 mi/h).



1 ft = 0.305 ft.
1 mi/h = 1.61 km/h

Figure 5. Comparison of sight intersection distance values calulated using modified AASHTO model (J=2.5 s) and Gap Acceptance model using varying values for G.

E. Design Element: Offset (Single) Left-Turn Lane Geometry, Signing, and Delineation

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Table 7. Cross-references of related entries for offset (single) left-turn lane geometry, signing, and delineation.

Applications in Standard Reference Manuals			
MUTCD (2000)	**AASHTO Green Book (1994)**	**NCHRP 279 Intersection Channelization Design Guide (1985)**	**Traffic Engineering Handbook (1999)**
Sect. 1A.13, *median, regulatory signs, road delineators, stop line*, & *wrong-way arrows* Sect. 1A.14, *Abbreviations* Table 2B-1 Sects. 2A.24, 2B.3, 2B.29, 2B.30, 2B.32, 2B.33 & 2E.50 Figs. 2A-2 through 2A-6, 2E-31 and 2E-32 Sect. 3B.4 Fig. 3B-11 a, b & d Fig. 3B-21 Sect. 3B.11 Sects. 3B.16 & 3B.19 Figs. 3B-19, 3B-21, 3B-22	Pg. 45, Para. 1 Pgs. 679-687, Sects. on *Island Size and Designation* & *Delineation and Approach-End Treatment* Pgs. 783-787, Sects. on *Median Left-Turn Lanes* & *Median End Treatment*	Pg. 1, 1st bullet Pg. 3, 2nd col., Para. 5 Pg. 6, Table 2-1 Pg. 10, Table 2-4 & 2nd col., Para. 3 Pg. 14, Sect. on *Decision Sight Distance* Pg. 17, Middle fig. Pg. 29, Para. 1 Pg. 34, Para. 1 and top fig. Pg. 35, Paras. 2-3	Pgs. 375-376, Sect. on *Intersection Sight Distance (ISD)* Pg. 386, Para. 4 Pg. 388, Para. 2

Sects. 3B.21, 3C.03, 3D.03, 3E.01, 3G.04 through 3G.06		Pg. 60, Middle fig.	

Studies examining older driver crashes and the types of maneuvers being performed just prior to the collision have consistently found this group to be overinvolved in left-turning crashes at both rural and urban signalized intersections and have indicated that failure to yield the right-of-way (as the turning driver) was the principal violation type (Staplin and Lyles, 1991; Council and Zegeer, 1992). Underlying problems for the maneuver errors include the misjudgment of oncoming vehicle speed, misjudgment of available gap, assuming the oncoming vehicle was going to stop or turn, and simply not seeing the other vehicle. Joshua and Saka (1992) noted that sight distance problems at intersections which result from queued vehicles in opposite left-turn lanes pose safety and capacity deficiencies, particularly for unprotected (permitted) left-turn movements. These researchers found a strong correlation between the offset for opposite left-turn lanes--i.e., the distance from the inner edge of a left-turn lane to the outer edge of the opposite left-turn lane--and the available sight distance for left-turning traffic.

The alignment of opposite left-turn lanes and the horizontal and vertical curvature on the approaches are the principal geometric design elements that determine how much sight distance is available to a left-turning driver. Operationally, vehicles in the opposite left-turn lane waiting to turn left can also restrict the (left-turning) driver's view of oncoming traffic in the through lanes. The level of blockage depends on how the opposite left-turn lanes are aligned with respect to each other, as well as the type/size of vehicles in the opposing queue. Restricted sight distance can be minimized or eliminated by offsetting opposite left-turn lanes so that left-turning drivers do not block each other's view of oncoming through traffic. When the two left-turn lanes are exactly aligned, the offset distance has a value of zero. Negative offset describes the situation where the opposite left-turn lane is shifted to the left. Positive offset describes the situation where the opposite left-turn lane is shifted to the right. Figure 6 illustrates the relationships between the opposite left-turn lanes for negative and positive offset lane geometry. Positive offset left-turn lanes and aligned left-turn lanes provide greater sight distances than negative offset left-turn lanes, and a positive offset provides greater sight distance than the aligned configuration.

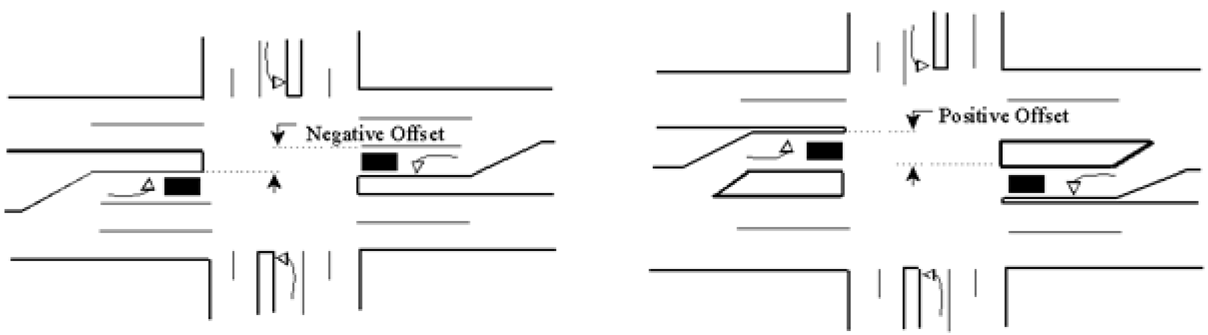


Figure 6. Relationship of left-turn lanes for negative and positive offset geometry.

Older drivers may experience greater difficulties at intersections as the result of diminished visual capabilities such as depth and motion perception, as well as diminished attention-sharing (cognitive) capabilities. Studies have shown that there are age differences in depth and motion perception. Staplin, Lococo, and Sim (1993) found that the angle of stereopsis (seconds of arc) required for a group of drivers age 75 and older to discriminate depth using a commercial vision tester was roughly twice as large as that needed for a group of drivers ages 18 to 55 to achieve the same level of performance. However, while accurate perception of the distance to geometric features delineated at intersections--as well as to potentially hazardous objects such as islands and other raised features--is important for the safe use of these facilities, relatively greater attention by researchers has been placed upon motion perception, where dynamic stimuli (usually other vehicles) are the primary targets of interest. It has been shown that older persons require up to twice the rate of movement to perceive that an object is approaching, and they require significantly longer to perceive that a vehicle is moving closer at a constant speed (Hills, 1975). A study investigating causes of older driver overinvolvement in turning crashes at intersections, building on the previously reported decline for detection of angular expansion cues, did not find evidence of overestimation of time-to-collision (Staplin et al., 1993). At the same time, a relative insensitivity to approaching (conflict) vehicle speed was shown for older versus younger drivers; this result was interpreted as supporting the notion that older drivers rely primarily or exclusively on perceived *distance*--not time or velocity--to perform gap acceptance judgments, reflecting a reduced ability to integrate time and distance information with increasing age. Thus, a principal source of risk at intersections is the error of an older, turning driver when judging gaps in front of fast vehicles.

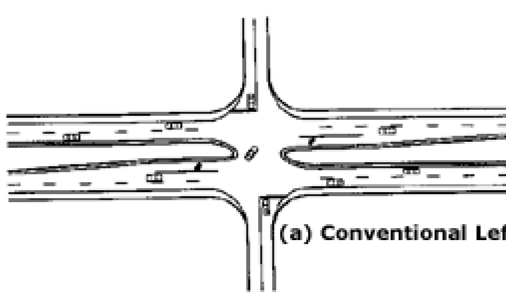
Several recent studies examining the minimum required sight distance for a driver turning left from a major roadway to a minor roadway, as a function of major road design speed, have provided data necessary to determine: (1) the left-turn lane offset value needed to achieve the minimum required sight distance; and (2) the offset value that will provide unlimited sight distance. A fundamental premise in these studies, which are described below, is that it is not the amount of left-turn lane offset *per se*, but rather the sight distance that a given level of offset provides that should be the focus of any recommendations pertaining to the design of opposite left-turn lanes.

In a study conducted by McCoy, Navarro, and Witt (1992), guidelines were developed for offsetting opposite left-turn lanes to eliminate the left-turn sight distance problem. All minimum offsets specified in the guidelines are positive. For 90-degree intersections on level tangent sections of four-lane divided roadways, with 3.6-m (12-ft) wide left-turn lanes in 4.9-m (16-ft) wide medians with 1.2-m (4-ft) wide medial separators, the following conclusions were stated by McCoy et al. (1992): (1) a 0.6-m (2-ft) positive offset provides unrestricted sight distance when the opposite left-turn vehicle is a passenger car, and (2) a 1.06-m (3.5-ft) positive offset provides unrestricted sight distance when the opposite left-turn vehicle is a truck, for design speeds up to 113 km/h (70 mi/h).

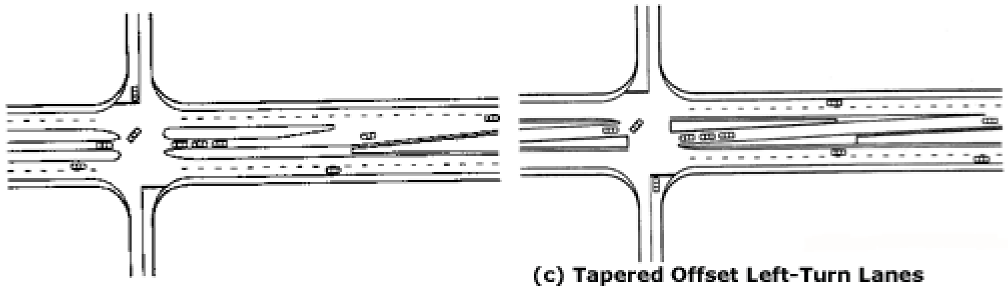
Harwood, Pietrucha, Wooldridge, Brydia, and Fitzpatrick (1995) conducted an observational field study and a crash analysis to develop design policy recommendations for the selection of median width at rural and suburban divided highway intersections based on operational and safety considerations. They found that at rural unsignalized intersections, both crashes and undesirable driving behaviors decrease as median width increases. However, at suburban signalized and unsignalized intersections, crashes and undesirable behaviors increase as the median width increases. At suburban intersections, it is therefore suggested that the median should not generally be wider than necessary to accommodate pedestrians and the appropriate median left-turn treatment needed to serve current and anticipated future traffic volumes. Harwood et al. stated that wider medians generally have positive effects on traffic operations and safety; however, wider medians can result in sight restrictions for left-turning vehicles due to the presence of opposite left-turn vehicles. The most common solution to this problem is to offset the left-turn lanes, using either parallel offset or tapered offset left-turn lanes.

Figure 7 compares conventional left-turn lanes with these two alternative designs. As noted by Harwood et al. (1995), parallel and tapered offset left-turn lanes are still not common, but are used increasingly to reduce the risk of crashes due to sight restrictions from opposite left-turn vehicles. Parallel offset left-turn lanes with 3.6-m (12-ft) widths can be constructed in raised medians with widths as narrow as 7.2 m (24 ft), and can be provided in narrower medians if restricted lane widths or curb offsets are used or a flush median is provided (Bonneson, McCoy, and Truby, 1993). Tapered offset left-turn lanes generally require raised medians of 7.2 m (24 ft) or more in width.

For separation of the left-turn lane from through traffic in alternative designs such as those discussed above, the practitioner must choose between raised channelization and channelization accomplished through the use of pavement markings. As noted earlier, left-turn channelization separating through and turning lanes may, because of its placement, constitute a hazard when a raised treatment is applied, especially on high-speed facilities. Detection and avoidance of such hazards requires visual and response capabilities known to decline significantly with advancing age, supporting recommendations for treatments to improve the contrast for these channelizing features at intersections (see Design Element C).



(a) Conventional Left-turn Lanes



(b) Parallel Offset Left-Turn Lanes (c) Tapered Offset Left-Turn Lanes

Figure 7. Alternative Left-turn treatments for rural and suburban divided highways.
Source: Bonneson, McCoy and Truby (1993).

As discussed in some detail under Design Element D, Staplin, Harkey, Lococo, and Tarawneh (1997) performed a laboratory study, field study, and sight distance analysis to measure driver age differences in performance under varying traffic and operating conditions, as a function of varying degrees of offset of opposite left-turn lanes at suburban arterial intersections. Research findings indicated that an increase in sight distance through positively offsetting left-turn lanes can be beneficial to left-turning drivers, particularly older drivers. In the field study, where left-turn vehicles needed to cross the paths of two or three lanes of conflicting traffic (excluding parking lanes) at 90-degree, four-legged intersections, four levels of offset of opposite left-turn lane geometry were examined. These levels include: (a) 1.8-m (6-ft) "partial positive" offset, (b) aligned (no offset) left-turn lanes, (c) 0.91-m (3-ft) "partial negative" offset, and (d) 4.3-m (14-ft) "full negative" offset. All intersections were located within a growing urban area where the posted speed limit was 56 km/h (35 mi/h). Additionally, all intersections were controlled by traffic-responsive semi-actuated signals, and all left-turn maneuvers were completed during the permitted left-turn phase at all study sites.

In the analysis of the field study lateral positioning data, it was found that the partial positive offset and aligned locations had the same effect on the lateral positioning behavior of drivers. Drivers moved approximately 1.5 m (5 ft) to the left when there was a large negative offset, clearly indicating that sight distance was limited. There was a significant difference between the partial negative offset geometry and the partial positive offset or aligned geometries, suggesting a need for longer sight distance when opposite left-turn lanes are even partially negatively offset. The fact that older drivers (and females) were less likely to position themselves (i.e., pull into the intersection) in the field studies highlights the importance ofproviding adequate sight distance for unpositioned drivers, for all left-turn designs. Vehicle positioning refers to the location within an intersection at which a left-turning vehicle waits for an acceptable gap in the opposing through traffic stream; specifically, at issue is the positioning behavior of drivers attempting to make a left turn through the conflicting through traffic while being opposed or blocked by at least one vehicle trying to make a left-turn maneuver from the opposite direction. The restriction of sight distance for an unpositioned versus a positioned driver at an intersection with aligned left-turn lanes is shown in figure 8.

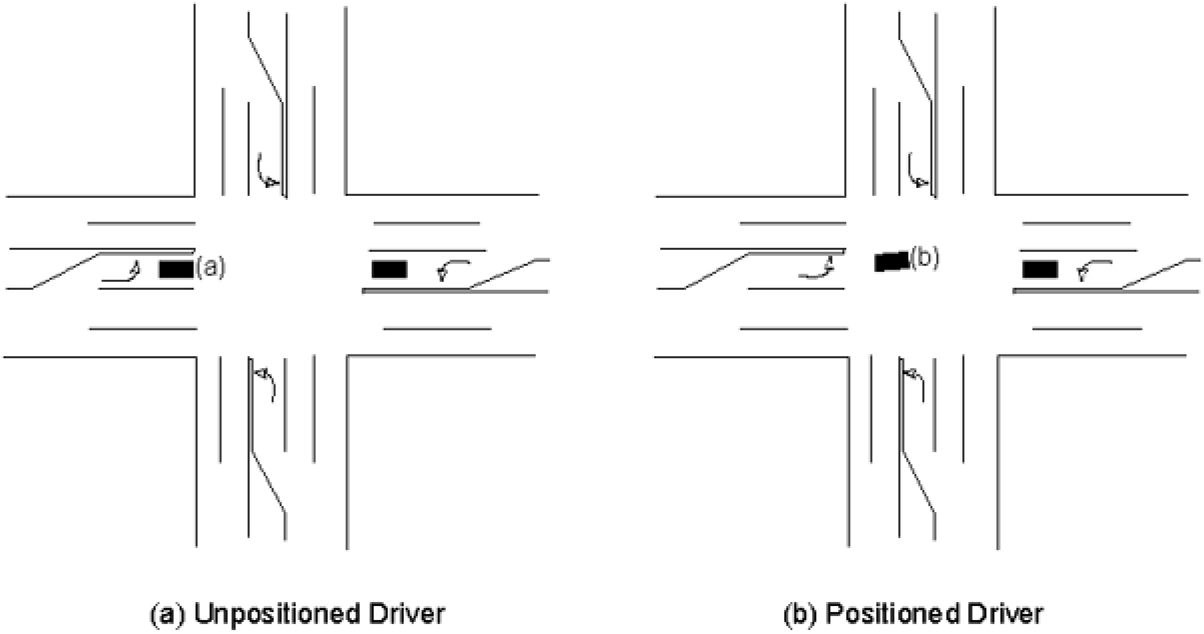
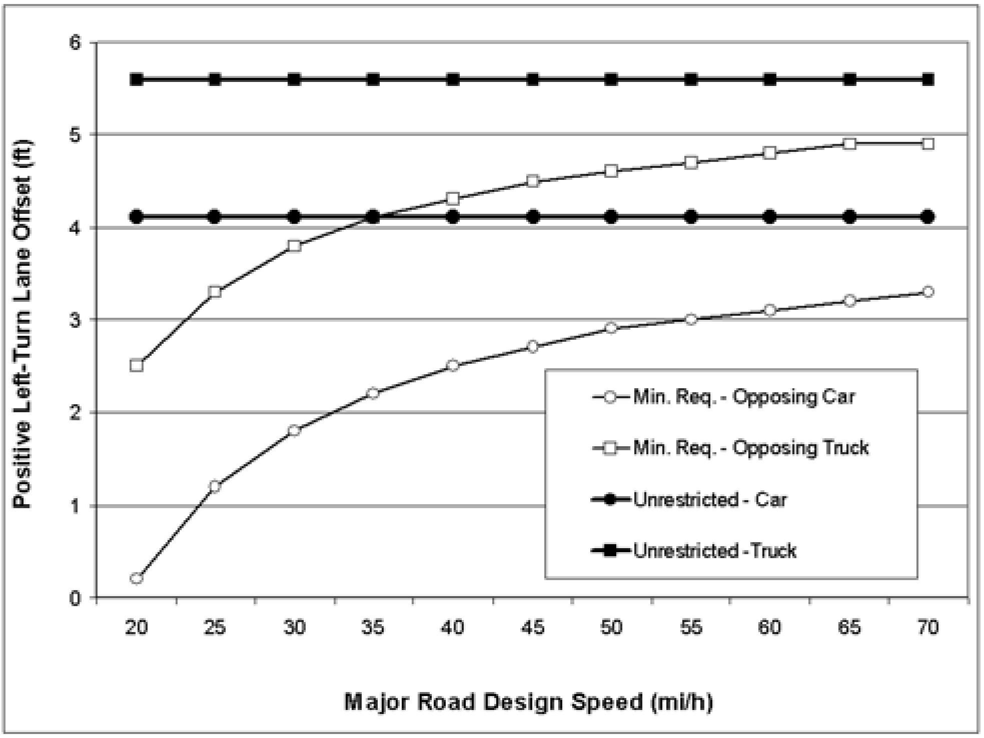


Figure 8. Difference in sight-distance restriction for an unpositioned driver and a positioned driver at an aligned intersection with an opposing left-turning driver.

Several issues were raised in the research conducted by Staplin et al. (1997) regarding the adequacy of the current and proposed intersection sight distance (ISD) models for a driver turning left from a major roadway. The researchers exercised alternative sight distance models, including the current AASHTO Case V model using 2.0 s for perception-reaction time (PRT), a modified AASHTO model using a 2.5-s PRT, and a Gap Acceptance model proposed in NCHRP 383 by Harwood, Mason, Brydia, Pietrucha, and Gittings (1996). The proposed Gap Acceptance model relies on a critical gap value in place of PRT and maneuver time. A detailed description of the model parameters and output can be found in the FHWA report entitled *Intersection Geometric Design and Operational Guidelines for Older Drivers and Pedestrians* (Staplin et al., 1997). Of particular significance was the finding that the modified AASHTO model with the longer PRT of 2.5 s was the model most predictive of actual field operations. Also of significance was the dramatic decrease in required sight distance that occurred with the gap acceptance model compared with the traditional AASHTO model. Across all intersections and all design speeds, the required sight distance was approximately 23 percent less using the gap acceptance model. However, this was expected since the rationale behind the use of a gap acceptance model (cf. Harwood et al., 1996), in place of the current AASHTO model, is the fact that drivers are commonly observed accepting shorter gaps than those implied by the current model. As discussed under Design Element D, subsequent analyses established a recommendation for use of an 8.0-s gap size (plus 0.5 s for each additional lane crossed) to adjust the Gap Acceptance to accommodate older driver needs for increased sight distance.

Regardless of which model is used to compute ISD for drivers turning left off a major roadway, a practical countermeasure to increase the sight distance is through positive offset of left-turn lanes. As shown in the study by Staplin et al. (1997), such designs result in significantly better performance on the part of all drivers, but especially for older drivers. Prior work by McCoy et al. (1992) examined the issue of offset left-turn lanes, and developed an approach that could be used to compute the amount of offset that is required to minimize or eliminate the sight restriction caused by opposing left-turn vehicles.

This approach, incorporating the parameters represented in the intersection diagram shown earlier in figure 4 (see Design Element D), was applied to the intersections in the study by Staplin et al. (1997) to determine the amount of offset that would be required when using the modified AASHTO model (i.e., J = 2.5 s). The left-turn lane offsets required to achieve the minimum required sight distances calculated using this model are shown in figure 9, in addition to the offsets required to provide unrestricted sight distance. Based on intersections examined in the study, the offset necessary to achieve unrestricted sight distance for opposing left-turning cars is 1.2 m (4.1 ft) and for opposing left-turning trucks is 1.7 m (5.6 ft).



1 ft = 0.305 ft.
1 mi/h = 1.61 km/h

Figure 9. Left-turn lane offset design values necessary to achieve unrestricted sight distances calculated using either the modified AASHTO model (J= 2.5 s) *or* the Gap Acceptance Model with G=8.0s.

Finally, the potential for wrong-way maneuvers, particularly by older drivers, at intersections with positive offset channelized left-turn lanes was raised during a panel meeting comprised of older driver experts and highway design engineers, during the conduct of the research performed by Staplin et al. (1997). The concern expressed was that drivers turning left from the minor road may turn too soon and enter the channelized left-turn lane, instead of turning around both medians. Similar concern was raised by highway engineers surveyed by Harwood et al. (1995) during the conduct of NCHRP project 15-14(2). These authors reported that the potential for wrong-way movements by opposing-direction vehicles entering the left-turn roadway is minimal if proper signing and pavement markings are used.

Researchers studying wrong-way movements at intersections--particularly the intersection of freeway exits with secondary roads--have found that such movements resulted from left-turning vehicles making an early left turn rather than turning around the nose of the median, and have proposed and tested several countermeasures. Scifres and Loutzenheiser (1975) reported that indistinct medians are design elements that reduce a driver's ability to see and understand the overall physical and operational features of an intersection, increasing the frequency of wrong-way movements. They suggested delineation of the median noses to increase their visibility and improve driver understanding of the intersection design and function. Also, increasing the conspicuity of ONE WAY, WRONG WAY, and DO NOT ENTER signs by using larger-than-standard (MUTCD) size signs, and using retroreflective sheeting on these signs that provides for high brightness at the wide observation angles typical of the sign placements and distances at which these signs are viewed (e.g., 1.0+ degrees) will be of benefit to drivers, particularly those with age-related diminished visual and attentional capabilities. Parsonson and Marks (1979) found that the use of the two-piece, 7.1-m-(23.5-ft-) long arrow pavement marking (wrong-way arrow) was effective in preventing wrong-way entries onto freeway exit ramps in Georgia. Later work in this State found a benefit of pulling the nose back from the intersection, and extending the median line from the nose to the intersection using painted markings and raised retroreflectors; this treatment reduced the frequency of impacts with the median by turning vehicles, particularly trucks.[(1)]

F. Design Element: Treatments/Delineation of Edgelines, Curbs, Medians, and Obstacles

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Table 8. Cross-references of related entries for treatments/delineation of edgelines, curbs, medians, and obstacles.

Applications in Standard Reference Manuals				
MUTCD (2000)	**AASHTO Green Book (1994)**	**Roadway Lighting Handbook (1978)**	**NCHRP 279 Intersection Channelization Design Guide (1985**	**Traffic Engineering Handbook (1999)**
Sect. 1A.13, *edgeline markings, island,* & *object markers* Sect. 3A.06 Sects. 3B.09, 3B.10, 3B.11, 3B.13, & 3B.21 Sects. 3C.01 through 3C.03 Sect. 3E.01 Sect. 3F.02 Sects. 3G.01 through 3G.06	Pg. 45, Para. 1 Pg. 314, Para. 7 Pg. 315, Para. 1 Pgs. 344-348, Sect. on *Types of Curbs* Pg. 347, Para. 5 Pg. 348, Paras. 1-3 Pg. 475, Para. 6 Pg. 519, Para. 2 Pg. 637, Para. 7 Pg. 639, Fig. IX-7a Pgs. 679-689, Sects. on *Island Size and Designation, Delineation and Approach-End Treatment*, & *Right-Angle Turns With Corner Islands* Pg. 755, Sect. on *Shape of Median End* Pgs. 756 & 761-763, Figs. IX-59 through IX-62 Pg. 783, Paras. 2-4 Pgs. 785-786, Figs. IX-73 & IX-74 Pgs. 786-787, Sect. on *Median End Treatment*	Pg. 2, 2nd col, Para. 1 Pg. 3, Para. 4 Pg. 4, 1st bullet Pg. 9, Sect. on *Contrast* Pg. 17, Form 1 Pg. 21, Table 1 Pg. 24, Example Form 1 Pgs. 29-30, Sect. on *Adverse Geometry and Environment Warrant* *Pg. 31, Item, A*	Pg. 24, Para. 1 Pg. 35, Para. 2 & bottom left fig. Pg. 39, All figs. Pg. 66, 2nd col., Para. 1 Pgs. 69 & 75, Sects. on *Traffic Islands* & *Guide-lines for Selection of Island Type* Pg. 74, Fig. 4-31 Pg. 76, Item 1 Pgs. 102-103, Intersct. No. 8	Pg. 434, Sect. on *Edge (Fog) Lines* Pg. 436, Para. 2 Pg. 438, Item 5 Pg. 439, Sect. on *Obstruction Approach* Pg. 440, Paras. 5 &7.

The discrimination at a distance of gross highway features, as opposed to the fine detail contained in a sign message, governs drivers' perceptions of intersection geometric elements. Thus, the conspicuity of such elements as curbs, medians, and obstacles, as well as all raised channelization, is of paramount importance in the task of safely approaching and choosing the correct lane for negotiating an intersection, as well as avoiding collisions with the raised surfaces. During the conduct of their driving task analysis, McKnight and Adams (1970a, 1970b) identified five driving tasks related specifically to the conspicuity of intersection geometric elements: (1) maintain correct lateral lane position; (2) survey pavement markings; (3) survey physical boundaries; (4) determine proper lane position for the intended downstream maneuver; and (5) search for path guidance cues. The visual/perceptual requirement common to the performance of these tasks is contrast sensitivity: for detecting lane lines, pavement word and symbol markings, curbs and roadway edge features, and median barriers.

Older drivers' decreased contrast sensitivity, reduced useful field of view, increased decision time--particularly in response to unexpected events--and slower vehicle control movement execution combine to put these highway users at greater crash risk when approaching and negotiating intersections. The smaller the attentional demand required of a driver to maintain the correct lane position for an intended maneuver, the greater the attentional resources available for activities such as the recognition and processing of traffic control device messages and detection of conflict vehicles and pedestrians.

A variety of conspicuity-enhancing treatments are mandated in current practice. The MUTCD (section 3B.10, Approach Markings for Obstructions) specifies that pavement markings shall be used to guide traffic away from fixed objects (such as median islands and channelization islands) within a paved roadway. Section

3B.21 (Curb Markings) states that retroreflective solid yellow markings should be placed on the curbs of islands that are located in the line of traffic flow where the curb serves to channel traffic to the right of the obstruction, and that retroreflective solid white markings should be used (on curbs) when traffic may pass on either side of the island. Section 3E.01 (Colored Pavements) describes the use of colored pavements as traffic control devices, where yellow shall be used for median islands and white for channelizing islands, and section 3G.03 (Island Marking Application) describes the use of pavement and curb markings; object markers; and delineators for island marking application. Supplementary treatments, and requirements for in-service brightness levels for certain elements contained in existing guidelines, are presently at issue.

The conspicuity of curbs and medians, besides aiding in the visual determination of how an intersection is laid out, is especially important when medians are used as pedestrian refuges. Care must be taken to ensure that pedestrian refuges are clearly signed and made as visible as possible to passing motorists.

Research findings describing driver performance differences directly affecting the use of pavement markings and delineation focus upon (age-related) deficits in spatial vision. In a pertinent laboratory study conducted by Staplin, Lococo, and Sim (1990), two groups of subjects (ages 19-49 and 65-80) viewing a series of ascending and descending brightness delineation targets were asked to report when they could just detect the direction of roadway curvature at the horizon (roadway heading)--left versus right--from simulated distances of 30.5 m (100 ft) and 61 m (200 ft). Results showed that the older driver group required a contrast of 20 percent higher than the younger driver group to achieve the discrimination task in this study.

To describe the magnitude of the effects of age and visual ability on delineation detection/recognition distance and retroreflective requirements for threshold detection of pavement markings, a series of analyses using the Ford Motor Company PC DETECT computer model (cf. Matle and Bhise, 1984) yielded the stripe contrast requirements shown in table 9. PC DETECT is a headlamp seeing-distance model that uses the Blackwell and Blackwell (1971, 1980) human contrast sensitivity formulations to calculate the distance at which various types of targets illuminated by headlamps first become visible to approaching drivers, with and without glare from opposing headlights. The top 5 percent of 25-year-olds (the best-performing younger drivers) and bottom 5 percent of 75-year-olds (the worst-performing older drivers) were compared in this analysis.

Blackwell and Taylor (1969) conducted a study of surface pavement markings employing an interactive driving simulator, plus field evaluations. They concluded that driver performance --measured by the probability of exceeding lane limits--was optimized when the perceived brightness contrast between pavement markings and the roadway was 2.0. A study by Allen, O'Hanlon, and McRuer (1977) also concluded that delineation contrast should be maintained above a value of 2.0 for adequate steering performance under clear night driving conditions. In other words, because contrast is defined as the difference between target and background luminance, divided by the background luminance alone, these studies have asserted that markings must appear to be *at least* three times as bright as the road surface. Also, because these studies were not specifically focused on the accommodation of older drivers--particularly the least capable members of this group--the contrast requirements defined in more recent modeling studies and analyses, as presented in table 9, are accorded greater emphasis. Taking the indicated value for the least capable 5 percent of 75-year-olds into account, as well as the prior field evaluations, a contrast requirement of 3.0 for pavement markings appears most reasonable.

Important note: Whether luminance is measured in metric or English units [candelas per square meter (cd/m^2) or footlamberts (fL)], contrast is a dimensionless number; thus the present recommendations as well as the calculation of contrast level are independent of the unit of measure.

Table 9. Contrast requirements for edgeline visibility at 122 m (400 ft) with 5-s preview at a speed of 88 km/h (55 mi/h), as determined by PC DETECT computer model.

Driver age group/ % accommodated	Worst-case glare	No glare
Age 25 / top 5%	0.11	0.05
Age 75 / bottom 5%	7.21	3.74

Finally, inadequate conspicuity of raised geometric features at intersections has been brought to the attention of researchers during the conduct of several focus group studies involving older drivers. Subjects reported difficulty knowing where to drive, due to missing or faded roadway lines on roadway edges and delineation of islands and turning lanes. They also reported hesitating during turns, because they did not know where to aim the vehicle (Staplin, Lococo, and Sim, 1990). In another focus group, subjects suggested that the placement of advance warning pavement markings be located as far in advance of an intersection as practicable (Council and Zegeer, 1992). Drivers ages 66-77 and older participating in focus group discussions conducted by Benekohal, Resende, Shim, Michaels, and Weeks (1992), reported that intersections with too many islands are confusing because it is hard to find which island the driver is supposed to go around. Raised curbs that are unmarked are difficult to see, especially in terms of height and direction, and result in people running over them. These older drivers stated that they would prefer to have rumble strips in the pavement to warn them of upcoming concrete medians and to warn them about getting too close to the shoulder. In more recent focus group discussions conducted to identify intersection geometric design features that pose difficulty for older drivers and pedestrians (Staplin, Harkey, Lococo, and Tarawneh, 1997), drivers mentioned that they have problems seeing concrete barriers in the rain and at night, and characterized barriers as "an obstruction waiting to be hit."

An inventory of the materials and devices commonly employed to delineate roadway edges, curbs, medians, and obstacles includes: retroreflective paint or tape, raised pavement markers (RPM's), post-mounted delineators (PMD's), object markers, and chevron signs.

G. Design Element: Curb Radius

Table 10. Cross-references of related entries for curb radius.

Applications in Standard Reference Manuals			
AASHTO Green Book (1994)	**NCHRP 279 Intersection Channelization Design Guide (1985)**		**Traffic Engineering Handbook (1999)**
Pgs. 665-672, Sect. on *Effect of Curb Radii on Turning Paths* Pg. 675, Para. 2 Pgs. 752-755, Sect. on *Control Radius for Minimum Turning Paths* Pgs. 758-763, Sect. on *Median Openings Based on Control Radii for Design Vehicles Noted*	Pg. 1, 2nd bullet Pg. 6, Paras. 4-5 Pg. 10, Table 2-4 Pg. 20, Bottom fig. Pg. 21, 2nd col, Item 4 & Fig. 3-1 Pg. 22, 2nd fig from bottom of pg. Pg. 23, Bottom right fig. Pg. 26, Bottom fig. Pg. 35, Para. 2 Pg. 36, Middle fig. & associated notes Pg. 38, Middle fig. & associated notes Pgs. 66-69, Sects. on *Corner Radius Design* & *Radius of Turn*	Pgs. 70-73, Figs. 4-27 through 4-29 Pg. 77, Fig. 4-32 Pg. 83, Sects. on *Driveways Along Major Arterials and Collectors* & *Consideration of Pedestrians* Pgs. 84-87, Figs. 4-37 through 4-39 Pgs. 93-94, Intersct. No. 2 Pgs. 96-97, Intersct. No. 4 Pgs. 122-125, Intersct. Nos. 18-19 Pgs. 128-129, Intersct. No. 20B Pgs. 132-135, Intersct. Nos. 22-23	Pg. 355, Paras. 5-6 Pg. 356, Table 11-5 Pg. 358, Table 11-6 Pg. 387, Sect. on *Corner Radius Design* Pg. 409, Para. 3

Recommendations for this design element address the radius of the curve that joins the curbs of adjacent approaches to an intersection. The size of the curve radius affects the size of vehicle that can turn at the intersection, the speed at which vehicles can turn, and the width of intersection that must be crossed by pedestrians. If curb radii are too small, lane encroachments resulting in traffic conflicts and increased crash potential can occur. If the radii are too large, pedestrian exposure may be increased (although, if large enough, refuge islands may be provided). The procedures used in the design of curb radii are well detailed in the Green Book (AASHTO, 1994).

McKnight and Stewart (1990) identified the task of positioning a vehicle in preparation for turning as a critical competency. A significant problem identified in a task analysis to prioritize older drivers' problems with intersections is carrying out the tight, right-turn maneuver at normal travel speed on a green light (Staplin, Lococo, McKnight, McKnight, and Odenheimer, 1994). The problems are somewhat moderated when right turns are initiated from a stop, because the turn can be made more slowly, which reduces difficulties with short radii. Older drivers may seek to increase the turning radius by moving to the left before initiating the turn, often miscommunicating an intent to turn left and encouraging following drivers to pass on the right. Or, they may initiate the turn from the correct position, but swing wide into a far lane in completing the turn in order to lengthen the turning radius and thus minimize rotation of the steering wheel. Encroaching upon a far lane can lead to conflict with vehicles approaching from the right or, on multilane roads, oncoming drivers turning to their left at the same time. The third possibility is to cut across the apex of the turn, possibly dragging the rear wheels over the curb. Each of these shortcomings in lanekeeping can be overcome by a channelized right-turn lane or wider curb radii.

Chu (1994) found that relative to middle-aged drivers (ages 25-64), older drivers (age 65 and older) tend to drive larger automobiles and drive at slower speeds. Although large heavy cars are associated with a crash fatality rate that is less than one-quarter of that associated with the smallest passenger cars (Insurance Institute for Highway Safety, 1991) and are, therefore, a wise choice for older drivers who are more frail than their middle-aged counterparts, large vehicles have larger turning radii, which may exacerbate the problems older drivers exhibit in lanekeeping during a turn. Roberts and Roberts (1993) reported that common arthritic illnesses such as osteoarthritis, which affects more than 50 percent of the elderly population, and rheumatoid arthritis, affecting 1 to 2 percent, are relevant to the tasks of turning and gripping the steering wheel. A hand deformity caused by either osteoarthritis or rheumatoid arthritis may be very sensitive to pressure, making the driver unwilling to apply full strength to the steering wheel or other controls. In an assessment of 83 drivers with arthritis, Cornwell (1987) found that 83 percent of the arthritic group used both hands to steer, 7 percent used the right hand only, and 10 percent the left hand only; in this study, more than one-half of the arthritic group required steering modifications, either in the form of power steering or other assistive device such as a smaller steering wheel.

The *Intersection Channelization Design Guide* (Neuman, 1985) states that intersections on high-speed roadways with smooth alignment should be designed with sufficient radii to accommodate moderate- to high-speed turns. At other intersections, such as in residential neighborhoods, low-speed turns are desirable, and smaller corner radii are appropriate in these cases. Additionally, selection of a design vehicle is generally based on the largest standard or typical vehicle type that would regularly use the intersection. For example, a corner radius of 15 m (50 ft) will accommodate moderate-speed turns for all vehicles up to WB-50 (combination truck/large semitrailer with an overall length of 17 m [55 ft]). However, many agencies are designing intersections along their primary systems to accommodate a 21-m (70-ft), single trailer design vehicle (C-70). Table 4-8 (p. 66) of the *Intersection Channelization Design Guide* provides guidelines for the selection of a design vehicle. It further specifies in table 4-9 what the operational characteristics are of various corner radii. For example, a corner radius of less than 1.5 m (5 ft) is not appropriate even for P design vehicles (passenger cars), whereas a corner radius of 6-9 m (20-30 ft) will accommodate a low-speed turn for P vehicles, and a crawl-speed turn for SU vehicles (single unit truck, 9 m [30 ft] in length) with minor lane encroachment.

Of equal importance to the consideration of the right-turning design vehicle in determining curb radii is a consideration of pedestrian crossing time, particularly in urban areas. Smaller corner radii (less than 9 m [30 ft]) can decrease right-turn speeds and reduce open pavement area for pedestrians crossing the street. A consideration of vehicle turning speed and pedestrian crossing distance can contribute to the safe handling of vehicle/pedestrian crossing conflicts (Neuman, 1985). Hauer (1988) noted that "the larger the curb-curve radius, the larger the distance the pedestrian has to cover when crossing the road. Thus, for a sidewalk whose centerline is 1.8 m (6 ft) from the roadway edge, a 4.5-m (15-ft) corner radius increases the crossing distance by only 1 m (3 ft). However, a 15-m (50-ft) radius increases this distance by 8 m (26 ft), or 7 s of additional walking time." Hauer further stated that the following are widely held concerns with the widening of curb radii: (1) the longer the crossing distance, the greater the hazard to pedestrians, even though there may be space for refuge islands when the corner radius is large enough; (2) larger curb radii may induce drivers to negotiate the right turn at a higher speed; and (3) the larger the radius, the wider the turn, which makes it more difficult for the driver and the pedestrian to see each other. For these reasons, the safety of older persons at intersections, particularly pedestrians, may be adversely affected when large curb radii are provided.

In focus group discussions with 46 drivers ages 65-74 (young-old group) and 35 drivers age 75 and older (old-old group), 74 percent of drivers in each age group reported that tight intersection corner radii posed difficulty in maneuvering through the intersection for the following reasons: (1) there are visibility problem with sharp corners; (2) drivers sometimes hit curbs and median barriers; and (3) with sharp turns, trucks turning left into the adjacent opposing traffic lane end up face-to-face with drivers, requiring them to back up (Staplin, Harkey, Lococo, and Tarawneh, 1997). Approximately 24 percent of the young-old drivers and 34 percent of the old-old drivers suggested that medium rounding is sufficient to facilitate turning maneuvers and is safer than very broadly rounded corners because the latter encourages high-speed turns.

In a design preference study using slides to depict varying radii of corner curb cuts, four alternative curb geometries were presented to 30 drivers ages 65-74 (young-old group) and 30 drivers age 75 and older (old-old group) (Staplin et al., 1997). The four alternative geometries (depicted in figure 10) were: (1) a simple circular radius of 5.5 m (18 ft); (2) a simple circular radius of 12 m (40 ft); (3) a simple circular radius of 14.6 m (48 ft); and (4) a three-sided/truncated curve with the center side measuring 16.5 m (54 ft).

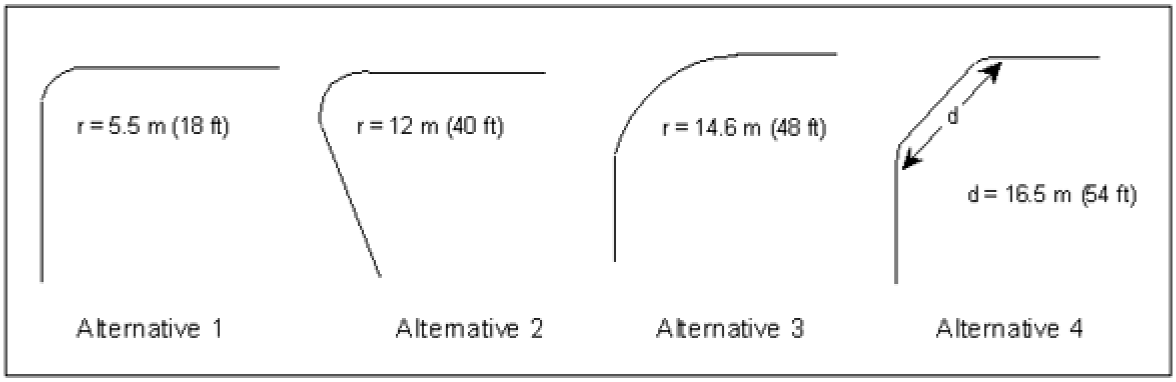

Figure 10. Alternative curb radii evaluated in laboratory preference study conducted by Staplin et al. (1997).

The alternatives were identically ranked by both older samples: Alternative 3 was consistently preferred, Alternative 4 placed second, Alternative 2 placed third, and Alternative 1 was least preferred. Both young-old and old-old drivers in this study were most concerned about ease of turning, citing the better maneuverability and less chance of hitting the curb as their primary basis of response. The second most common--but also strongly weighted--reason for the preference responses of both groups related to the degree of visibility of traffic on intersecting roadways, possibly explaining the slight preference for Alternative 2 over Alternative 1. Alternatives 3 and 4 both are described by corner curbline geometries offering ease of turning and good visibility; however, isolated responses to the truncated corner geometry (Alternative 4) indicated concerns that providing *too much* room in the right-turn path might result in a lack of needed guidance information and could lead to a maneuver error, and that it could be harder to detect pedestrians with this design.

In a field study conducted as part of the same project, three intersections providing right-turn curb radii of 12.2 m, 7.6 m, and 4.6 m (40 ft, 25 ft, and 15 ft) were evaluated to examine the effects of curb radii on the turning paths of vehicles driven by drivers in three age groups. One hundred subjects divided across three age groups drove their own vehicles around test routes using the local street network in Arlington, VA. The three age groups were "young/middle-aged" (ages 25-45), which contained 32 drivers; "young-old" (ages 65-74), containing 36 drivers; and "old-old" (age 75 and older), containing 32 drivers. The speed limit was 56 km/h (35 mi/h) and all intersections were located on major or minor arterials within a growing urban area. Data were only collected for turns executed on a green-signal phase.

Analysis of the free-flow speeds showed that all factors (age, gender, and geometry), and their interactions, were significant. Mean free-flow speeds were highest at the largest (12.2-m [40-ft]) curb radius location, for all age groups. A consistent finding showed that the slowest mean free-flow speeds were measured at the 4.6-m (15-ft) curb radius location for all age groups. Thus, larger curb radii increased the turning speeds of all drivers, with young/middle-aged and young-old drivers traveling faster than old-old drivers when making right turns.

H. Design Element: Traffic Control for Left-Turn Movements at Signalized Intersections

Table 11. Cross-references of related entries for left-turn movements at signalized intersections.

Applications in Standard Reference Manuals

MUTCD (2000)	AASHTO Green Book (1994)	NCHRP 279 Intersection Channelization Design Guide (1985)	Traffic Engineering Handbook (1999)
Sect. 1A.13, *approach, intersection, lane-use control signal, regulatory signs, sign legend,* & *traffic control signal,* Sect. 1A.14, *Abbreviations* Table 2B-1 Sects. 2B.17 through 2B.21, 2B.40, 3B.08 & 3B.18 Figs. 3B-11b, 3B-11c, 3B-20, & 3B-21 Sect. 4D.4 Sect. 4D.06 Sects. 4D.07, 4D.08, 4D.12, 4D.15, 4D.16, & 4D.18 Sect. 4J.02 Sects. 4J.03 through 4J.04	Pg. 319, Para. 2 Pg. 637, Paras. 6-8 Pgs. 639-640, Figs. IX-6 & IX-7 Pg. 641, Para. 1 Pg. 847, Para. 1 Pg. 848, Fig. X-17 Pgs. 852-860, Sect. on *Single Point Diamond*	Pg. 1, Item 3, 4th bullet Pg. 3, 2nd col., Para. 3 Pg. 21, Fig. 3-1 Pg. 28, Top fig. Pg. 29, Top left fig. Pg. 34, Top fig. & associated notes Pg. 37, Top left fig. Pg. 48, Para. 5 & Table 4-3 Pg. 49, Paras. 1, 2, & 4 and 2nd col, item 2 Pg. 54, Fig. 4-16, bottom left photo Pg. 57, Sects. on *Double Left-Turn Lanes--Guidelines for Use* & *Design of Double Left-Turn Lanes* Pgs. 58-60, Figs. 4-20 & 4-21 Pgs. 100-101, Intersct. No. 7 Pgs. 104-119, Intersct. Nos. 9-16 Pgs. 132-135, Intersct. Nos. 22-23 Pg. 144, Intersct. No. 33 Pgs. 148-151, Intersct. Nos. 35-36	Pg. 241, Paras. 6 & 9 Pg. 242, Para. 3 Pg. 316, Para. 7 Pgs. 332-333, Sect. on *Storage Lengths* Pg. 386, Para. 3 Pg. 427, Para. 3 Pg. 435, Sect. on *Stop Bars* Pg. 454, 4th Bullet Pg. 467, 2nd & 3rd Bullets Pgs. 470-479, Sects. on *Controller Phasing for Left Turns*, *Operational Modes,* & *Criteria for Determining Need and Mode* Pg. 515, Sect. on *Application to Left-Turn Lanes* Pgs. 522-524, Sect. on *Lane-Use Control Signals*

Crash analyses have shown that older drivers, ages 56-75 and age 76 and older, are overinvolved in left-turn maneuvers at signalized intersections, with failure to yield right-of-way or disregarding the signal the principal violation types (Staplin and Lyles, 1991; Council and Zegeer, 1992). Old-elderly drivers (age 75 and older) were more likely than younger drivers (ages 30-50) to be involved in left-turn crashes at urban signalized intersections, and both young-elderly (ages 65-74) and old-elderly were more likely to be involved in left-turn crashes at rural signalized intersections. In both cases, the crash-involved older drivers were more likely to be performing a left-turn maneuver than the younger drivers. In addition, Stamatiadis, Taylor, and McKelvey (1991) found that the relative crash involvement ratios for older drivers were higher at two-phase (no turning phase) signalized intersections than for multiphase (includes turn arrow) signalized intersections. This highlights problems older drivers may have determining acceptable gaps and maneuvering through traffic streams when there is no protective phase. Further, crash percentages increased significantly for older drivers when an intersection contained flashing controls, as opposed to conventional (red, yellow, green) operations. In this analysis, the greatest crash frequency at signalized intersections occurred on major streets with five lanes, followed closely by roadways containing four lanes. These configurations were most often associated with low-speed, high-volume urban locations, where intersection negotiation requires more complex decisions involving more conflict vehicles and more visually distracting conditions. Not surprisingly, Garber and Srinivasan's (1991) analysis of 7,000 intersection crashes involving drivers ages 50-64 and age 65 and older, found that the provision of a protected left-turn phase will aid in reducing the crash rates of the elderly at signalized intersections.

The change in the angular size of a moving object, such as an approaching vehicle observed by a driver about to turn left at an intersection, provides information crucial to gap judgments (i.e., speed and distance). Age-related declines (possibly exponential) in the ability to detect angular movement have been reported. Older persons may in fact require twice the rate of movement than younger persons to perceive that an object is approaching, given a brief (2.0 s) duration of exposure. Also, older persons participating in laboratory studies have been observed to require significantly longer intervals than younger persons to perceive that a vehicle was moving closer at constant speed: at 31 km/h (19 mi/h), decision times increased 0.5 s between ages 20 and 75 (Hills, 1975).

Compounding this age-related decline in motion perception, some research has indicated that, relative to younger subjects, older subjects underestimate approaching vehicle speeds (Hills and Johnson, 1980). Specifically, Scialfa, Guzy, Leibowitz, Garvey, and Tyrrell (1991) showed that older adults tend to overestimate approaching vehicle velocities at lower speeds and underestimate at higher speeds, relative to younger adults. Staplin, Lococo, and Sim (1993), while investigating causes of older driver overinvolvement in turning crashes at intersections, did not find evidence of overestimation of time-to-collision by older drivers in their perception of the closing distance between themselves and another vehicle approaching either head-on or on an intersecting path. However, a relative insensitivity to approach (conflict) vehicle speed was shown for older versus younger drivers, in that younger drivers adjusted their gap judgment of the "last safe moment" to proceed with a turn appropriately to take higher approach speeds into account, while older drivers as a group failed to allow a larger gap for a vehicle approaching at 96 km/h (60 mi/h) than for one approaching at 48 km/h (30 mi/h). The interpretation of this and other data in this study was that older drivers rely primarily or exclusively on perceived vehicle separation distance to reach maneuver "go/no go" decisions, reflecting a reduced ability to integrate time and distance information with increasing age. Thus, a principal source of risk at intersections is the error of an older, turning driver in judging gaps in front of fast vehicles.

Aside from (conflict vehicle) motion detection, an additional concern is whether there are age differences in how well drivers understand the rules under which the turns will be made--that is, whether older drivers have disproportionately greater difficulty in understanding the message that is being conveyed by the signal and any ancillary (regulatory) signs. If the signals and markings are not understood, at a minimum there may be delay in making a turn or, in the worst case, a crash could result if a protected operation is assumed where it does not exist.

A driver comprehension analysis conducted in a laboratory setting with drivers 30-60 years of age and older showed that green displays (those with the circular green indication alone, green arrow alone, or combinations of circular green and green arrow on the left-turn signal) were correctly interpreted with widely varying frequency, depending on the signals shown for the turning and through movements (Curtis, Opiela, and Guell, 1988). In most cases, performance declined as age increased; older drivers were correct approximately half as often as the youngest drivers. Most driver errors, and especially older driver errors, indicated signal display interpretations that would result in conservative behavior, such as stopping and/or waiting. A summary of the results follows. Overall, green arrows were better understood than circular green indications. Conversely, red and yellow arrows were less comprehensible than circular red and circular yellow indications. Potentially unsafe interpretations were found for red arrow displays in protected-only operations. The yellow arrow display was more often treated as a last chance to complete a turn when compared with a circular yellow indication. Driver errors were most frequent in displays that involved flashing operations, and multiple faces with different colors illuminated on the left-turn signal head, and in particular, different colors on the turn and through signals.

More specifically, Curtis et al. (1988) found that the circular green indication under permitted control was correctly interpreted by approximately 60 percent of the subjects. For protected-only operations, the green arrow (with circular red for through movement) was correctly answered by approximately 75 percent of drivers. For protected/permitted operation, the circular green alone was correctly answered by only 50 percent of the respondents, while the green arrow in combination with the circular green had approximately 70 percent correct responses. When the circular green with the green arrow was supplemented by the R10-12 sign LEFT TURN YIELD ON GREEN ·, only 34 percent of drivers answered correctly. This test result suggests that the MUTCD recommended practice may result in some driver confusion, as test subjects answered correctly more often when the sign was *not* present, even when the effects of regional differences in familiarity with the sign were considered. Apparently reinforcing this notion, the Maryland State Highway Administration (MSHA, 1993) reported a higher rate of left-turn collisions at three intersections where the R10-12 sign was installed than at three intersections where the sign was not installed. Unfortunately, driver age was not a study variable; also, medians were present (only) at sites with the R10-12 signs, and differences between sites in terms of signal phasing, traffic volumes and delays, and alignment and other aspects of intersection geometry, though noted, were not described. Other researchers have found improved driver comprehension with the use of the R10-12 sign, compared to other messages informing drivers of the decision rule for protected/permitted operations, as described later in this section.

When Hummer, Montgomery, and Sinha (1990) evaluated motorists' understanding of left-turn signal alternatives, they found that the protected-only signal was by far the best understood, permitted signals were less understood, and the protected/permitted the least understood. When a circular green for through traffic and a green arrow for left turns were displayed, the protected signal was clearly preferred over the permitted and protected/permitted signals, and the leading signal sequence was preferred more often than the lagging sequence. Respondents stated that the protected-only signal caused less confusion, was safer, and caused less delay than the permitted and protected/permitted signals. It should be noted, however, that while older persons were in the sample of drivers studied, they made up a very small percentage (8 of 402) and differences were hard to substantiate.

More recently, Knoblauch, Nitzburg, Reinfurt, Council, Zegeer, and Popkin (1995) examined the lack of understanding associated with a variety of protected and permitted left-turn signal displays. They found that many drivers, both younger and older, do not understand the protected/permitted signal phasing, and they suggested that efforts to improve motorist comprehension of left-turn signal phasing should be targeted at the entire driving population. In focus group discussions, many older drivers reported that they avoid intersections that do not have a protected-only phase or those where the time allowance for left turns was too short. In addition, the situation where the green arrow eventually turns to a circular green was generally confusing and not appreciated by the older participants. Among the recommendations made by the older drivers were:

- Provide as many protected left-turn opportunities as possible.
- Standardize the sequence for the left-turn green arrow so that it *precedes* solid green or red.
- Lengthen the protected left-turn signal.
- Lengthen the left-turn storage lanes so that turning traffic does not block through traffic.
- Make traffic signal displays more uniform across the United States, including the warning or amber phase.
- Standardize the position and size of signals.
- Provide traffic lights overhead and to the side at major intersections.
- Paint a yellow line in the pavement upstream of the signal in a manner that, if the driver has not reached the line before the light has turned yellow, he/she cannot make it through before the red light.
- Provide borders (backplates) around lights to minimize the effects of glare.
- Eliminate holiday decorations located overhead at intersections, because they are often green and red and may be confusing near signal faces.

Bonneson and McCoy (1994) also found a decreased understanding of protected and permitted left-turn designs with increased age, in a survey conducted in Nebraska with 1,610 drivers. In this study, the overlap phase (left-turn green arrow and through circular green illuminated) was the least understood by drivers wishing to turn left, with only one-half of the respondents answering correctly; most of the respondents who erred chose the safer course of action, which was to wait for a gap in oncoming traffic. In terms of signal head location, 4 to 5 percent more drivers were able to understand the protected/permitted display when it was centered in the left-turn lane (exclusive) as opposed to having the head located over the lane line (shared). Although the difference was statistically significant, Bonneson and McCoy point out that the difference may be too small to be of practical significance. In terms of lens arrangement, significantly more drivers understood both the permitted indication and the protected/MUTCD indication (left-turn green arrow and through circular red) in vertical and horizontal arrangements than in the cluster arrangement. Comparisons between the protected/MUTCD indication and a modified protected indication (green arrow with no circular red), showed that for the horizontal protected/permitted designs, 25 percent more drivers were able to understand the protected indication when the circular red was *not* shown with the green arrow, and for the vertical and cluster protected/permitted designs, 12 percent more drivers understood the modified protected indication. The point is that from an operational perspective, hesitancy as a result of misunderstanding will decrease the level of service and possibly result in crash situations.

An analysis of sign use by Bonneson an McCoy (1994) compared the exclusive cluster lens arrangement over the left-turn lane and exclusive vertical lens arrangement over the through lanes with and without the use of an auxiliary sign (LEFT TURN YIELD ON GREEN). Overall, the results indicated that driver understanding of the display increased by about 6 percent when there was *no* sign, though a closer examination of these data revealed that the specific operation signaled by the display was critical. *For the permitted indication, the sign appeared to help driver understanding*, whereas during the overlap and protected indications it appeared to confuse drivers.

Numerous studies have found that: (1) protected left-turn control is the safest, with protected/permitted being less safe than protected, but safer than permitted (Fambro and Woods, 1981; Matthais and Upchurch, 1985; Curtis et al., 1988); and (2) transitions from protected-only operations to protected/permitted operations experience crash increases (Cottrell and Allen, 1982; Florida Section of Institute of Transportation Engineers, 1982; Cottrell, 1985; Warren, 1985; Agent, 1987). According to Fambro and Woods (1981), for every left-turn crash during a protected phase, 10 would have occurred without protection. Before-and-after studies where intersections were changed from protected to permitted control have shown four- to seven-fold increases in left-turn crashes (Florida Section of Institute of Transportation Engineers, 1982; Agent, 1987).

Williams, Ardekani, and Asante (1992) conducted a mail survey of 894 drivers in Texas to assess motorists' understanding of left-turn signal indications and accompanying auxiliary signs. Drivers older than age 65 had the highest percentage of incorrect responses (35 percent). Results of the various analyses are as follows: (1) the use of a green arrow for protected-only left turns produced better comprehension than the use of a circular green indication, even when the circular green indication was accompanied by an auxiliary sign; (2) for a five-section signal head configuration, the display of a green left-turn arrow in isolation produced better driver understanding than the simultaneous display of a circular red indication and a green left-turn arrow; (3) the LEFT TURN YIELD ON GREEN · auxiliary sign was associated with the smallest percentage of incorrect responses, compared with the LEFT TURN ON GREEN AFTER YIELD sign, the PROTECTED LEFT ON GREEN sign, and the LEFT TURN SIGNAL sign; and (4) the percentage of incorrect responses was 50 percent lower in the presence of a circular red indication compared with a red arrow; the red arrow was often perceived to indicate that a driver may proceed with caution to make a permitted left turn.

In another study conducted by Curtis et al. (1988), it was found that the Delaware flashing red arrow was not correctly answered by any subject. The incorrect responses indicated conservative interpretations of the signal displays which would probably be associated with delay and may also be related to rear-end collisions. Drivers interpreted the Delaware signal as requiring a full stop before turning, because a red indication usually means "stop," even though the signal is meant to remind motorists to exercise caution but not necessarily to stop unless opposing through traffic is present. Hulbert, Beers, and Fowler (1979) found a significant difference in the percentage of drivers younger than age 49 versus those older than age 49 who chose the correct meaning of the red arrow display. Sixty-one percent of the drivers older than age 49 chose "no turning left" compared with 76 percent of those younger than age 49. Although other research has concluded that the left-turn arrow is more effective than the circular red in some left-turn situations in particular jurisdictions where special turn signals and exclusive turn lanes are provided (Noel, Gerbig, and Lakew, 1982), drivers of all ages will be better served if signal indications are consistent. Therefore, it is recommended that the use of the arrow be reserved for protected turning movements and the color red be reserved for circular indications to mean "stop."

Hawkins, Womak, and Mounce (1993) surveyed 1,745 drivers in Texas to evaluate driver comprehension of selected traffic control devices. The sample contained 88 drivers age 65 and older. Three alternative signs describing the left-turn decision rule were evaluated: (1) R10-9, PROTECTED LEFT ON GREEN ARROW (in the Texas MUTCD but not the national MUTCD); (2) R10-9a, PROTECTED LEFT ON GREEN (in the Texas MUTCD but not the national MUTCD); and (3) R10-12, LEFT TURN YIELD ON GREEN . The R10-12 sign did the best job of the signs in the survey informing the driver of a permitted left-turn condition, with 74.5 percent choosing the desirable response. Of those who responded incorrectly, 13.6 responded that they would wait for the green arrow, and 4.3 percent made the dangerous interpretation that the left turn was protected when the circular green was illuminated. Incorrect responses were more often made by drivers age 65 and older.

The decisional processes drawing upon working memory crucial to safe performance at intersections may be illustrated through a study of alternative strategies for presentation of left-turn traffic control messages (Staplin and Fisk, 1991). This study evaluated the effect of providing advance left-turn information to drivers who must decide whether or not they have the right-of-way to proceed with a protected turn at an intersection. Younger (mean age of 37) and older (mean age of 71) drivers were tested using slide animation to simulate dynamic approaches to intersection traffic control displays, with and without advance cueing of the "decision rule" (e.g., LEFT TURN MUST YIELD ON GREEN ·) during the intersection approach. Without advance cueing, the decision rule was presented only on a sign mounted on the signal arm across the intersection as per standard practice, and thus was not legible until the driver actually reached the decision point for the turning maneuver. Cueing drivers with advance notice of the decision rule through a redundant upstream posting of sign elements significantly improved both the accuracy and latency of all drivers' decisions for a "go/no go" response upon reaching the intersection, and was of particular benefit to the older test subjects. Presumably, the benefit of upstream "priming" is derived from a reduction in the requirements for serial processing of concurrent information sources (sign message and signal condition) at the instant a maneuver decision must be completed and an action performed.

Stelmach, Goggin, and Garcia-Colera (1987) found that older adults were particularly impaired when preparation was not possible, showing disproportionate response slowing when compared with younger subjects. When subjects obtained full information about an upcoming response, reaction time (RT) was faster in all age groups. Stelmach et al. (1987) concluded that older drivers may be particularly disadvantaged when they are required to initiate a movement in which there is no opportunity to prepare a response. Preparatory intervals and length of precue viewing times are determining factors in age-related differences in movement preparation and planning (Goggin, Stelmach, and Amrhein, 1989). When preparatory intervals are manipulated in a way that older adults have longer stimulus exposure and longer intervals between stimuli, they profit from the longer inspection times by performing better and exhibiting less slowness of movement (Eisdorfer, 1975; Goggin et al., 1989). Since older drivers benefit from longer exposure to stimuli, Winter (1985) proposed that signs should be spaced farther apart to allow drivers enough time to view information and decide what action to take. Increased viewing time will reduce response uncertainty and decrease older drivers' RT.

Differences in maneuver decisions reported by Staplin and Fisk (1991) illustrate both the potential problems older drivers may experience at intersections due to working memory deficits, and the possibility that such consequences of normal aging can to some extent be ameliorated through improved engineering design practices. Staplin and Fisk (1991) also showed that older drivers had higher error rates and increased decision latencies for situations where the left turn was not protected. In particular, the most problematic displays were those with only one steady illuminated signal face (circular green) accompanied by a sign that indicated that it was *not* safe to proceed into the intersection with the assumption of right-of-way (LEFT TURN YIELD ON GREEN ·). A correct response to this combination depends on the inhibition of previously learned "automatic" responses; a signal element with one behavior (go) was incorporated into a traffic control display requiring another, conflicting behavior.

Hummer, Montgomery, and Sinha (1991) evaluated leading and lagging signal sequences using a survey of licensed drivers in Indiana, an examination of traffic conflicts, an analysis of crash records, and a simulation model of traffic flow, to evaluate motorists' understanding and preference for leading and lagging schemes as well as determining the safety and delay associated with each scheme. Combinations of permitted and protected schemes included: (1) protected-only/leading, in which the protected signal is given to vehicles turning left from a particular street before the circular green is given to the through movement on the same street; (2) protected-only/lagging, in which the green arrow is given to left-turning vehicles after the through movements have been serviced; (3) protected/permitted, in which protected left turns are made in the first part of the phase and a circular green indication allows permitted turns later in the phase; and (4) permitted/protected, in which permitted turns are allowed in the first part of the phase and protected left turns are accommodated later in the phase. The protected-only/leading and protected/permitted schemes are known as "leading," and the protected-only/lagging and permitted/protected are known as "lagging" schemes. Of the 402 valid responses received, 248 respondents preferred the leading, 59 preferred the lagging sequence, and 95 expressed no preference. The most frequent reasons given for preference of the leading sequence were: it is more like normal; it results in less delay; and it is safer. There are apparent tradeoffs here, however; the leading sequence was associated with a higher conflict rate with pedestrians and a higher rate of run-the-red conflicts (drivers turning left during the clearance interval for opposing traffic), while the intersections with a lagging sequence were associated with a significantly higher rate of indecision conflicts than the leading intersections due to violations in driver expectancy. Overall, it is judged that consistency in signal phasing across intersections within a jurisdiction, as well as across jurisdictions, should be a priority, and that use of a leading protected left-turn phase offers the most benefits. A discussion of countermeasures for the protection of pedestrians may be found in the material that presents the Rationale and Supporting Evidence for Design Elements I and P.

Upchurch (1991) compared the relative safety of 5 types of left-turn phasing using Arizona Department of Transportation crash statistics for 523 intersection approaches, where all approaches had a separate left-turn lane, 329 approaches had 2 opposing lanes of traffic, and 194 approaches had 3 opposing lanes. The five types of left-turn phasing included (1) permitted, (2) leading protected/permitted, (3) lagging protected/permitted, (4) leading protected-only, and (5) lagging protected-only. For the 495 signalized intersections in the State highway system, most samples represented a 4-year crash history (1983-1986). For the 132 signalized intersections in 6 local jurisdictions in Arizona, samples ranged from 4 months to 4 years, all between 1981 and 1989. When the crash statistics were stratified by various ranges of left-turn volume and various ranges of opposing volume (vehicles per day), the following observations and conclusions were made for sample sizes greater than five, eliminating any conclusions about lagging protected-only phasing:

- Leading protected-only phasing had the lowest left-turn crash rate in almost every case. This was true in every left-turn volume range and every opposing volume range except one (19 out of 20 cases). Lagging protected/permitted was the exception for 3 opposing lanes and left-turn volumes of 0-1,000.
- When there were two lanes of opposing traffic, lagging protected/permitted tended to have the worst crash rate.
- When there were three lanes of opposing traffic, leading protected/permitted tended to have the worst crash rate.
- When there were two lanes of opposing traffic, the order of safety (crash rate from best to worst) was leading protected-only, permitted, leading protected/permitted, and lagging protected/permitted. However, there was a small difference in the crash rate among the last three types of phasing.
- When there were three lanes of opposing traffic, the order of safety (crash rate from best to worst) was leading protected-only, lagging protected/permitted, permitted, and leading protected/permitted.

Upchurch (1991) compared the crash experience of 194 intersections that had been converted from one type of phasing to another in a simple before-and-after design. For each conversion, 4 years of before-crash data and 4 years of after-crash data were used, where available. At approaches having *two* opposing lanes of traffic, the statistics for conversions from permitted to leading protected/permitted and vice versa reinforced each other, suggesting that leading protected/permitted is safer than permitted. At approaches having *three* opposing lanes of traffic, the statistics for conversions from leading protected-only to leading protected/permitted and vice versa reinforced each other, suggesting that leading protected-only is safer than leading protected/permitted

Parsonson (1992) stated that a lagging left-turn phase should be used only if the bay provides sufficient storage, as any overflow of the bay during the preceding through-movement will spill into the adjacent through lane, blocking it. A lag should also be reserved for those situations in which opposing left-turn movements (or U-turns) are safe from the left-turn trap (or are prohibited). The "left-turn trap" occurs when the left-turning driver's right-of-way is terminated, while the opposing (oncoming) approach continues with a green arrow and an adjacent through movement. Thus, left-turning drivers facing a yellow indication are trapped; they believe that the opposing traffic will also have a yellow signal, allowing them to turn on the yellow or immediately after. Since the opposing traffic is not stopping, the turning driver is faced with a potentially hazardous situation. Locations where the left-turn trap is not a hazard include T-intersections, and those where the left turn (or U-turn) opposing the green arrow is prohibited or is allowed only on a green arrow (protected-only phasing). In addition, driver expectancy weighs heavily in favor of leading left turns, and driver confusion over lagging left turns results in losses in start-up time.

I. Design Element: Traffic Control for Right-Turn/RTOR Movements at Signalized Intersections

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Table 12. Cross-references of related entries for traffic control for right turn/RTOR movements at signalized intersections.

Applications in Standard Reference Manuals			
MUTCD (2000)	**AASHTO Green Book (1994)**	**NCHRP 279 Intersection Channelization Design Guide (1985)**	**Traffic Engineering Handbook (1999)**
Sect. 1A.13, *intersection, right-of-way [assignment], sign legend,* & *traffic control signal (traffic signal)* Sect. 1A.14, *Abbreviations* Table 2B-1 Sects. 2B.17 through 2B.21, 2B.40 Sects. 3B.08 & 3B.19 Figs. 3B-11b, 3B-20, 3B-21 Sects. 4D.04, 4D.05, 4D.07, 4D.08 Sects. 4D.10 through 4D.12, 4D.15, 4D.16, & 4D.18	Pg. 319, Para. 2 Pg. 534, Para. 1 Pg. 641, Para. 1 Pg. 718, Paras. 1-2	Pg. 3, 2nd col, Para. 2 Pg. 37, Para. 2 & top right fig. Pgs. 61-65, Sect. on *Exclusive Right-Turn Lanes* Pg. 100-101, Intersct. No. 7 Pgs. 106-113, Intersct. Nos. 10-13 Pgs. 124-125, Intersct. No. 19 Pgs. 132-135, Intersct. Nos. 22-23 Pgs. 148-149, Intersct. No. 35	Pg. 239-242, Sect. on *Turn Restrictions* Pgs. 332-333, Sect. on *Storage Lengths* Pg. 384, Item 7. Pg. 386, Paras. 2 & 6 Pg. 461, Sect. on *Right-Turn Guidelines for Warrant Application* Pgs. 522-524, Sect. on *Lane-Use Control Signals*

The right-turn-on-red (RTOR) maneuver provides increased capacity and operational efficiency at a low cost (Institute of Transportation Engineers [ITE], 1999). However, traffic control device violations and limited sight distances need to be addressed in order to reduce the potential for safety problems. ITE concluded that a significant proportion of drivers do not make a complete stop before executing an RTOR, and a significant portion of drivers do not yield to pedestrians. In a review of the literature on RTOR laws and motor vehicle crashes, Zador (1984) reported findings that linked RTOR to a 23 percent increase in all right-turning crashes, a 60 percent increase in pedestrian crashes, and a 100 percent increase in bicyclist crashes. Analysis of police crash reports in four States indicated that drivers who are stopped at a red light are looking left for a gap in traffic and do not see pedestrians and bicyclists coming from their right (Preusser, Leaf, DeBartolo, and Levy, 1982). Eldritch (1989) noted that, adding to the adverse effects RTOR has on pedestrian crashes, many motorists persist in making right turns on red even when there is a sign that prohibits the maneuver.

The most recent data available on the safety impact of RTOR were provided by Compton and Milton (1994) in a report to Congress by the National Highway Traffic Safety Administration. Using Fatal Analysis Reporting System (FARS) data and data from four State files for 1989-1992, it was concluded that RTOR crashes represented a small proportion of the total number of traffic crashes in the four States (0.05 percent) and of all fatal (0.03 percent), injury (0.06 percent), and signalized-intersection crashes (0.40 percent). FARS data showed that approximately 84 fatal crashes per year occurred involving a right-turning vehicle at an intersection where RTOR is permitted; however, because the status of the traffic signal indication is not available in this database, the actual number of fatal crashes that occurred when the signal was red is not known. Slightly less than one-half of these crashes involved a pedestrian (44 percent), 10 percent involved a bicyclist, and 33 percent involved one vehicle striking another. Although no data on the age of the drivers involved in RTOR crashes were provided, there are reasons for concern that increasing problems with this maneuver may be observed with the dramatic growth in the number of older drivers in the United States.

The difficulties that older drivers are likely to experience making right turns at intersections are a function of their diminishing gap-judgment abilities, reduced neck/trunk flexibility, attention-sharing deficits, slower acceleration profile, and their general reduction in understanding traffic control devices compared with younger drivers. Right-turning drivers face possible conflicts with pedestrians, and restrictions in the visual attention of older drivers may allow pedestrian and vehicular traffic to go unnoticed. The fact that pedestrians may be crossing the side street, where they enter the path of the right-turning vehicle, places a burden upon the driver to search the right-turning path ahead. The result is the need to share attention between oncoming vehicles approaching from the left and pedestrians in the path to the right. Limitations in the range of visual attention, frequently referred to as "useful field of vision," further contribute to the difficulty of older drivers in detecting the presence of pedestrians or other vehicles near the driver's path. Older drivers, who may have greater difficulty maintaining rapid eye movements and associated head movements, are less likely to make correct judgments on the presence of pedestrians in a crosswalk or on their walking speed (Habib, 1980).

Researchers have identified that the right-turn maneuver is more problematic for older drivers compared with young or middle-aged drivers, presumably as a result of age-related diminished visual, cognitive, and physical capabilities. Knoblauch, Nitzburg, Reinfurt, Council, Zegeer, and Popkin (1995) conducted an analysis of right-angle, left-turning, right-turning, side-swipe, and rear-end crashes at intersections in Minnesota and Illinois for the time period of 1985-1987, comparing crash proportions and characteristics of "middle-aged" drivers ages 30-50, "young-elderly" drivers ages 65-74, and "old-elderly" drivers age 75 and older. Turning right accounted for 35.8, 39.3, and 42.9 percent, respectively, of the middle-aged, young-elderly, and old-elderly drivers' crashes at urban locations. It appears that, for right-turning crashes, the middle-aged driver is most likely crossing the intersection on a green signal and the older drivers are turning right on a red signal in front of the oncoming middle-aged driver. Similar patterns emerged from examination of the rural signalized-intersection precrash maneuvers, with middle-aged drivers most often traveling straight, and older drivers most often turning left or right. Looking at the contributing factors in angle and turning collisions for both rural and urban signalized locations, the middle-aged group was much more likely to be characterized by the police officer as having exhibited "no improper driving." This occurred in 65 percent of the crashes involving this age group, compared with 30.7 percent of the young-elderly, and 13.4 percent of the old-elderly. The two elderly groups were more likely to be cited for failing to yield (42.0 percent of the old-elderly, 31.9 percent of the young-elderly, and 10.9 percent of the middle-aged); disregarding the traffic control device (30.7 percent of the old-elderly, 22.0 percent of the young-elderly, and 10.3 percent of the middle-aged); and driver inattention (8.2 percent of the old-elderly, 8.9 percent of the young-elderly, and 6.4 percent of the middle-aged).

Knowledge testing has indicated that, compared with younger drivers, older drivers are less familiar with the meaning of traffic control devices and relatively new traffic laws (McKnight, Simone, and Weidman, 1982). "Newness" of traffic laws, in this regard, relates not to the period of time that has elapsed since the device or law was implemented, but the low frequency with which drivers come in contact with the situation. Older drivers may not encounter right turn on red after stop (RTOR), no turn on red (NTOR), or red right-turn arrow situations on a daily basis, due to the significantly lower amount and frequency of driving in which they are engaged. The demonstrated lack of understanding for the red right-turn arrow (Hulbert, Beers, and Fowler, 1979) and increased violations associated with this display (Owolabi and Noel, 1985) would be of particular concern for older road users, drivers and pedestrians alike.

Knoblauch et al. (1995) found that both drivers younger than the age of 65 and drivers age 65 and older failed to understand that they could turn right on a circular red after stopping in the right lane. Although the survey indicated that older drivers were more likely to stop and remain stopped (45 percent) than younger drivers (36 percent), the differences were not significant.

Staplin, Harkey, Lococo, and Tarawneh (1997) conducted a controlled field study to measure differences in drivers' RTOR behavior as a function of driver age and right-turn lane channelization. In this study, 100 subjects divided across 3 age groups were observed as they drove their own vehicles around test routes using the local street network in Arlington, Virginia. The three age groups were young/middle-aged (ages 25-45), young-old (ages 65-74), and old-old (age 75+). The percentage of drivers who made RTOR maneuvers at the four intersections was included as a measure of mobility.

Staplin et al. (1997) found that significantly fewer drivers in the old-old driver group attempted to make an RTOR (16 percent), compared with young/middle-aged drivers (83 percent) and young-old drivers (45 percent). Similarly, young/middle-aged drivers made an RTOR nearly 80 percent of the time when they had the chance to do so, compared with nearly 36 percent for the young-old drivers and 15 percent for the old-old drivers. Drivers made significantly fewer RTOR's at the skewed channelized intersection than at the other three locations. Analysis of the percentage of drivers who made an RTOR without a complete stop showed that age, right-turn lane geometry, gender, and the age-by-geometry interaction were significant. Young/middle-aged drivers made an RTOR without a complete stop nearly 35 percent of the time, compared with nearly 25 percent for the young-old and 3 percent for the old-old drivers. Channelized intersections with or without exclusive acceleration lanes encouraged making an RTOR without a complete stop. The nonchannelized and the skewed locations showed the lowest percentage of RTOR's without a complete stop, and were not significantly different from each other. The three age groups showed significantly different performance. Old-old drivers almost always stopped before making an RTOR regardless of the right-turn lane geometry. In only 1 of 26 turns did an older driver *not* stop before making an RTOR; this occurred at the channelized right-turn lane with an acceleration lane. At the nonchannelized intersection (which was controlled by a STOP sign), 22 percent of the young/middle-aged drivers, 5 percent of the young-old drivers, and none of the old-old drivers performed an RTOR without a stop. Where an acceleration lane was available, 65 percent of the young/middle-aged drivers continued through without a complete stop, compared with 55 percent of the young-old drivers and 11 percent of the old-old drivers. The increased mobility exhibited by the younger drivers at the channelized right-turn lane locations (controlled by YIELD signs) was not exhibited by old-old drivers, who stopped in 19 of the 20 turns executed at the channelized locations. In summary, with increases in driver age, the likelihood of RTOR decreases to a very low level for the present cohort of old-old drivers, but when these individuals do engage in this behavior, they are virtually certain to come to a complete stop before initiating the maneuver. Therefore, the emphasis is to ensure adequate sight distance for the older turning driver, to provide sign and signal indications that are most easily understood by this group, and to prompt these motorists to devote adequate attention to pedestrians who may be in conflict with their turning maneuver.

Zegeer and Cynecki (1986) found that offsetting the stop line--moving the stop line of adjacent stopped vehicles back from the intersection by 1.8 to 3.0 m (6 to 10 ft)--was effective in providing better sight distance to the left for RTOR motorists. It also reduced the RTOR conflicts with other traffic and resulted in more RTOR vehicles making a full stop behind the stop line. The offset stop line was recommended as a countermeasure for consideration at RTOR-allowed sites that have two or more lanes on an approach and heavy truck or bus traffic, or unusual geometrics.

Zegeer and Cynecki (1986) also found that a novel sign (circular red symbol with NO TURN ON RED, shown in figure 11) was more effective than the standard black-and-white NO TURN ON RED (R10-11a) sign, especially when implemented near the signal. This countermeasure resulted in an overall reduction in RTOR violations and pedestrian conflicts. They offered that the circular red symbol on the sign helps draw drivers' attention to it, particularly as intersections are associated with a preponderance of signs and information, and recommended that it should be added to the MUTCD as an alternate or approved as a replacement to the current R10-11a design. Increasing the size of the standard NO TURN ON RED sign from its present size of 600 mm x 750 mm (24 in x 30 in) to 750 mm x 900 mm (30 in x 36 in) reduced the proportion of violations at most of the test sites. Finally, Zegeer and Cynecki (1986) found that an electronic NO TURN ON RED blank-out sign was found to be slightly better than the standard MUTCD sign in terms of reducing violations, and it was effective in increasing RTOR maneuvers when RTOR was appropriate, thereby reducing vehicle delay. Although the sign is more expensive than standard signs and pavement markings, the authors concluded it may be justified in situations where pedestrian protection is critical during certain periods (i.e., school zones) or during a portion of the signal cycle when a separate, opposing left-turn phase may conflict with an unsuspecting RTOR motorist.

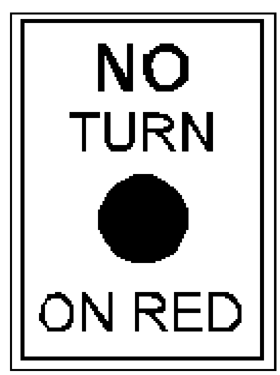

Figure 11. Novel sign tested as a countermeasure by Zegeer and Cynecki, 1986.

J. Design Element: Street-Name Signing

Table 13. Cross-references of related entries for street-name signing.

Applications in Standard Reference Manuals	
MUTCD (2000)	**AASHTO Green Book (1994)**
Sect. 1A.14, *Abbreviations* Sects. 2A.08, 2A.12, 2A.15, 2A.17, 2D.01 through 2D.06, 2D.38 & 2E.26	Pg. 45, Para. 1 Pg. 314, Paras 2-3

The MUTCD (1988) states that the lettering on street-name signs (D3) should be at least 100 mm (4 in) high. The MUTCD (2000) incorporates a change that specifies that the lettering on post-mounted street-name signs should be at least 150 mm (6 in) high, and that larger letters should be used for street-name signs that are mounted overhead. It provides an option for using 100 mm (4 in) lettering on street-name signs that are posted on local roads with speed limits 40 km/h (25 mi/h) or less. Burnham (1992) noted that the selection of letter size for any sign must evaluate the needs of the user, which are continuously changing as a function of changes in automotive technology, the roadway system, and the population itself. For example, Phoenix, Arizona, a city with a large older driver population, has been using "jumbo" street-name signs at signalized intersections since 1973. These signs are 400 mm (16 in) in height and use 200 mm (8 in) capital letters (*Rural and Urban Roads*, 1973). It is estimated that by the year 2020, 17 percent or more of the population will be older than 65 years of age, and by the year 2030, 1 in 5 Americans will be older than age 65 (U.S. Bureau of the Census, 1996). The ability to read street signs is dependent on visual acuity as well as divided attention capabilities, both of which decline significantly with advancing age.

Older drivers participating in focus groups and completing questionnaires for traffic safety researchers over the past decade have consistently stated that larger street signs with bigger lettering and standardization of sign placement overhead would make driving an easier task (Yee, 1985; Gutman and Milstein, 1988; Cooper, 1990; Staplin, Lococo, and Sim, 1990; Benekohal, Resende, Shim, Michaels, and Weeks, 1992; Knoblauch, Nitzburg, Reinfurt, Council, Zegeer, and Popkin, 1995). Problems with placement included signs that were either obstructed by trees, telephone poles, billboards, or large trucks, or placed too close to or across the intersection rather than on the near side. Older drivers stated that they needed more advance notice regarding upcoming cross streets and larger street-name signs placed overhead, to give them more time to make decisions about where to turn. Also noted were difficulties reading traffic signs with too much information in too small an area, and/or with too small a typeface, which results in the need to slow down or stop to read and respond to the sign's message. May (1992) noted that providing sufficient time to allow motorists to make appropriate turning movements when approaching cross streets can improve safety and reduce congestion, and that consistent street signing across political jurisdictions can be helpful in this regard, as well as presenting an orderly, predictable picture of the region to tourists, business people, and residents.

Taoka (1991) discussed "spare glance" duration in terms of how drivers allocate their visual search time among different tasks/stimuli. The tasks ranged from side/rearview mirror glances during turning to reading roadway name signs. Although specific results were not differentiated by age, Taoka asserted that 85th percentile glance times at signs (about 2.4 s) were likely too long, as 2.0 s is the maximum that a driver should divert from the basic driving task. Since older drivers are more apt to be those drivers taking longest to read signs, these results imply that they will commonly have problems dividing attention between searching for/reading signs and the basic driving task. Malfetti and Winter (1987) observed that older drivers exhibited excessive vehicle-braking behavior whenever a signal or road sign was sighted. This was categorized as an unsafe behavior, because it is confusing and disruptive to following traffic when the lead vehicle brakes for no apparent reason. These researchers obtained many descriptions of older drivers who stopped suddenly at unexpected times and in unexpected places, frequently either within the intersection or 12 m (40 ft) before the intersection to read street signs.

The visibility of retroreflective signs must be considered with regard to their dual requirements of detection and legibility. The sign components affecting detection are sign size, color, shape, brightness, and message or content design. External factors affecting sign detection include its placement (e.g., left, right, or overhead), the visual complexity of the area, and the contrast of the sign with its background. The component parts of retroreflective signs that determine legibility fall into two major classes of variables: character and message. Character variables include the variables related to brightness--i.e., contrast, luminance, color, and contrast orientation--as well as font, letter height, letter width, case, and stroke width. Message variables address the visibility issues of spacing and include interletter, interword, interline, and copy-to-border distances.

Most studies of sign legibility report legibility distance and the letter height of the stimulus; dividing the former measure by the latter defines the "legibility index" (LI), which can serve as a common denominator upon which to compare different studies. Forbes and Holmes (1939) used the LI to describe the relative legibility of different letter styles. Under daytime conditions, series B, C, and D were reported to have indexes of 0.4 m/mm, 0.5 m/mm, and 0.6 m/mm (33, 42.5, and 50 ft/in), respectively. Forbes, Moskowitz, and Morgan (1950) found the wider, series E letters to have an index of 0.66 m/mm (55 ft/in). Over time the value of 0.6 m/mm (50 ft/in) of letter height became the nominal, though arbitrary and disputed, standard. The LI is important to the size requirement determination for a sign in a specific application. Based on the physical attributes of the older driver population, the standard of 50 ft of legibility for every 1 in of letter height (corresponding to a visual acuity of 20/25) exceeds the visual ability of approximately 40 percent of the drivers between ages 65 and 74. The MUTCD (2000) section 2A.14, which provides guidance for determining sign letter heights, indicates that sign letter heights should be determined based on 25 mm (1 inch) of letter height per 12 m (40 ft) of legibility distance; this shift is certainly desirable considering the human factors issues addressed in this chapter.

Mace (1988), in his work on minimum required visibility distance (MRVD) for highway signs, noted the following relationships:

Required letter size = MRVD / LI *or* Required LI = MRVD / letter size

Either the letter size or the LI may be manipulated to satisfy the MRVD requirement, which specifies the minimum distance at which a sign should be read for proper driver reaction.

Olson and Bernstein (1979) suggested that older drivers should not be expected to achieve a LI of 0.6 m/mm (50 ft/in) under most nighttime circumstances. The data provided by this report gives some expectation that 0.48 m/mm (40 ft/in) is a reasonable goal under most conditions. A 0.48 m/mm (40 ft/in) standard can generally be effective for older drivers, given contrast ratios greater than 5:1 (slightly higher for guide signs) and luminance greater than 10 cd/m^2 for partially reflectorized signs. With regard to the effect of driver age on legibility, Olson, Sivak, and Egan (1983) concluded that older drivers require more contrast between the message and the sign's background than younger drivers to achieve the same level of comprehension. They also noted that legibility losses with age are greater at low levels of background luminance. A reduction in legibility distance of 10 to 20 percent should be assumed when signs are not fully reflectorized. (It should be noted that the MUTCD, 2000 includes text in section 2A.08 that states that regulatory, warning, and guide signs shall be retroreflective or illuminated to show the same shape and color by both day and night, unless specifically stated otherwise in the MUTCD text discussion of a particular sign or group of signs. Section 2D.03 further states that all messages, borders, and legends on guide signs shall be retroreflective, and all backgrounds shall be retroreflective or illuminated). Also, higher surround luminance improved the legibility of signs more for older drivers and reduced the negative effects of excessive contrast. In general, the LI for older drivers is 70 to 77 percent of the LI for younger drivers. The average LI for older drivers is clearly below the nominal value of 0.6 m/mm (50 ft/in) of letter height. The means for older drivers are generally between 0.48 m/mm and 0.6 m/mm (40 and 50 ft/in); however, the 85th percentile values reported are between 0.36 and 0.48 m/mm (30 and 40 ft/in) (Sivak, Olson, and Pastalan, 1981; Kuemmel, 1992; Mace, Garvey, and Heckard, 1994). Mace (1988) concluded that a most conservative standard would provide drivers with 2 minutes of arc, which corresponds to 20/40 vision and a 0.36 m/mm (30 ft/in) LI.

In a laboratory simulation study, Staplin et al. (1990) found that older drivers (ages 65-80) demonstrated a need for larger letter sizes to discern a message on a guide sign, compared with a group of younger drivers (ages 19-49). To read a one-word sign, older drivers required a mean letter size corresponding to 2.5 minutes of visual angle (or a Snellen acuity of 20/50), compared with the mean size required by younger drivers of 1.8 minutes of visual angle (or Snellen acuity of 20/35). Character size requirements increased for both age groups when the message contained four words, to 3.78 minutes of visual angle (acuity equivalent of 20/75) for the older drivers, and to 2.7 minutes of visual angle (acuity equivalent of 20/54) for the younger drivers. The main effect of age for the word and message legibility measure was highly significant. Staplin et al. (1990) concluded that for standard highway signing, an increase in character size in the range of 30 percent appears necessary to accommodate age-related acuity differences across the driving population.

Tranchida, Arthur, and Stackhouse (1996) conducted a field study using older drivers who drove the research laboratory's vehicle at nighttime, to determine the legibility distances of street-name signs as a function of sheeting type.

The subjects included 9 males ages 68 to 74, and 9 females ages 62 to 83. The four sheeting types were: Type IX, Type VII, Type III, and Type I (American Society for Testing and Materials, 2001). Intersections of three levels of complexity were used: high complexity/ high traffic activity (e.g., large intersection in downtown business area); intermediate complexity/intermediate traffic activity (e.g., small intersection area in suburban small business/apartment area); and low complexity/low traffic activity (e.g., residential area of single-family homes). All intersections were lighted. Street-name signs with invented names (Strike, Strong, Stress, Straw, Story, and Storm) were created using Series C letters, with a 152-mm (6- in) uppercase "S", followed by 112-mm (4.5-in) lowercase letters. There were no borders on the street-name signs. The signs were placed on the far side of the intersection, either on the right or the left side, and the drivers' task was to read aloud the street name as soon as it was legible to them, as they approached at a speed of 33 km/h (20 mi/h). The vehicle was a 2-door sedan with automatic transmission, power steering, and power brakes.

The mean legibility distances across the three intersections and two street sides were as follows for the four sheeting types: Type VII=51.8 m (170 ft); Type IX=52.5 m (172 ft); Type III=43.3 m (142 ft); and Type I=39.6 m (130 ft). Legibility distances were always longer for signs placed on the right side of the street than for those placed on the left. The mean legibility distances for the signs mounted on the right side of the road and corresponding luminances of the sheeting at the legibility distances are as follows: Type VII=76.2 m (205 ft) and 4.392 cd/m^2; Type IX=61.2 m (201 ft) and 7.369 cd/m^2; Type III=53.9 m (177 ft) and 1.1314 cd/m^2; and Type I=53 m (174 ft) and 0.9671 cd/m^2. Sheeting Types VII and IX performed similarly, and produced significantly longer legibility distances than both Type III and Type I sheeting. However, Types VII and Type IX provided significantly longer legibility distances only for the intersections with high complexity viewing conditions. There was no significant benefit in legibility distance for Type VII and Type IX sheeting at the two streets making up the low complexity intersection and on one street that was less traveled and less visually complex than the other in the intermediate complexity intersection.

These results suggest that at visually complex intersections with exaggerated demands for divided attention, the use of retroreflective sheeting that provides increased legibility distance would be of clear benefit to older drivers. Sheeting that provides for high retroreflectance overall, and particularly at wide observation angles typical when viewingstreet-name signs, would best meet this need. The anticipated benefit is that fewer glances will need to be directed toward the sign to determine the legend, and more effort can be devoted to vehicle control and visual search for traffic and pedestrian conflicts.

The use of mixed-case letters on overhead street-name signs was studied by Garvey, Gates, and Pietrucha (1997). Based on this research, it was recommended that for any approach with a 56 km/h (35 mi/h) or lower speed limit, an overhead street name sign should have 20-cm (8-in) uppercase and 15-cm (6-in) lowercase letters. For approaches with a speed limit above 56 km/h, an overhead street-name sign should contain 25-cm (10-in) uppercase and 20-cm (8-in) lowercase letters. This recommendation is based on the need for street name signs to be legible for 5.5 s before the intersection, which allows for a 1.5-s alerted perception-reaction time to read a sign and initiate a response (Johannson and Rumar, 1971), plus a 4.0-s interval to complete a combined speed reduction and tracking task (McGee, Moore, Knapp, and Sanders, 1979). Street-name signs should therefore be readable at 91 m (300 ft) at speeds of 56 km/h (35 mi/h), and at 137 m (450 ft) at 88 km/h (55 mi/h).

In an earlier study, Garvey, Meeker, and Pietrucha (1996) found a 12 to 15 percent increase in recognition distance for mixed-case text over all upper case legends under both daytime and nighttime conditions. However, this result was for recognition of words that drivers already knew would appear on the signs. Because the reading of street name signs is often a recognition task, rather than a pure legibility task, the reading distance of street name signs will be higher than would be predicted on driver visual acuity alone. At the same time, street-name legends provide useful information only when they can be read and understood by motorists. This fact underscores the focus on manipulations of those characteristics of sign legends that can increase reading distance. The rationale for mixed-case letters is reported above; the case for enhancements of street-name letter fonts follows. Another obvious manipulation, of course, is simply the size of the letters themselves.

Garvey, Pietrucha, and Meeker (1997) investigated an experimental font in two controlled field studies, using drivers ages 65 to 83. To accurately describe this research, it is necessary to use a trademarked name; however, this does not imply an endorsement of this product by the U.S. Government. Also, until this font undergoes the procedures required for MUTCD approval (rule making process), a recommendation cannot be made to use a non-standard font on standard highway signs. Garvey at al. (1997) compared the recognition distances and legibility distance of words displayed in mixed-case Clearview™ font with those displayed in Standard Highway Series D uppercase font, and mixed-case Standard Highway Series E(M) font. The Clearview™ font was developed to have open, wider spaces within a letter, to eliminate the effects of irradiation/halation that is caused by bright, bold stroke widths that "bleed" into a character's open spaces, rendering it illegible. Since each Clearview™ character has more openness than the Standard Highway font, the intercharacter spacing is smaller. Clearview^{TM} spacing results in words that take up 10.8 percent less space than Standard Highway fonts, such that a 12 percent increase in Clearview^{TM} character height results in words equal in sign space to words presented in the Standard fonts. The Clearview^{TM} font was produced in a regular version, with visual proportions similar to the Standard FHWA Series E(M) font, as well as in a condensed version, with visual proportions similar to the Standard FHWA Series D font. Two sizes of the Clearview^{TM} font were displayed: Clearview^{TM} 100 (fonts matched to Standard Highway font height) and Clearview^{TM} 112 (fonts 112 percent of Standard Highway font letter height, but equal in overall sign size to Standard Highway font). The fonts tested are described in table 14. The Clearview^{TM} fonts will be referred to as Clear Condensed 100, Clear Condensed 112, Clear 100, and Clear 112 throughout the remainder of this section. White words were created with either encapsulated lens (ASTM Type III:R_A=250 cd/lux/m^2) material or microprismatic sheeting designed for short-distance brightness (R_A=430 cd/lux/m^2), and were displayed on a green sign panel measuring 1.2 m^2 (4 ft^2). Each sign contained three place names, each containing six letters (from the same font). The study was conducted using one subject at a time, who was seated in the front passenger's seat of a vehicle driven by the experimenter. For each test run, the vehicle was started at a point 305 m (1,000 ft) from the sign.

For the word recognition study, the experimenter read aloud the place name that the subject was to look for on a sign. As the experimenter drove toward the sign at approximately 8 to 16 km/h (5 to 10 mi/h), the subject's task was to tell the experimenter when he or she could determine where the place name was located on the sign: top, middle, or bottom. The distance from the sign at which the subject answered correctly was recorded as the recognition distance. Twelve older drivers (mean age = 70.9 years) completed the word recognition study during the day, and another 12 older drivers (mean age = 74.8 years) completed the study at nighttime.

Table 14. Fonts tested by Garvey, Pietrucha, and Meeker (1997).

Font Name	Case	Letter Height
Clear Condensed 100	mixed case	Upper Case: 12.7 cm (5 in) Lower Case: 9.9 cm (3.9 in) loop height
Clear Condensed 112	mixed case	Upper Case: 14.2 cm (5.6 in) Lower Case: 11.2 cm (4.4 in)
Standard Highway Series D	uppercase	12.7 cm (5 in)
Standard Highway Series E(M)	mixed case	Upper Case: 12.7 cm (5 in) Lower Case: 9.9 cm (3.9 in) loop height
Clear 100	mixed case	Upper Case: 12.7 cm (5 in) Lower Case: 9.9 cm (3.9 in) loop height
Clear 112	mixed case	Upper Case: 14.2 cm (5.6 in) Lower Case: 11.2 cm (4.4 in)

A new set of 24 subjects was recruited for the legibility study, with half completing the study during daytime (mean age = 71.3 years) and half at nighttime (mean age = 73.9 years).

For the word legibility study, subjects were presented with only one word on a sign, and wererequired to read the word. Legibility distance was recorded at the point where subjects correctly read the word.

Results of the word recognition study indicated that during the daytime, there were no significant differences between either the Clear 100 or Clear 112 and the Series E(M) fonts. However, when comparing the Clear Condensed 100 and Clear Condensed 112 to the Series D font, the mixed-case fonts produced significantly longer recognition distances (14 percent greater) than the all uppercase Standard Highway font. At nighttime, the Clear 100 font did not produce recognition distances significantly different from those obtained with the Standard E(M) font, however, the Clear 112 font produced significantly greater recognition distances (16 percent greater) than the Standard E(M) font. The Clear 112 and Clear Condensed 112 fonts produced significantly longer recognition distances than the all-uppercase Series D font. Under both daytime and nighttime, there were no significant effects of material brightness, for the word recognition study. The mean daytime and nighttime recognition distances for the six fonts are displayed in table 15.

Table 15. Daytime and Nighttime Recognition Distances for Fonts Studied by Garvey, Pietrucha, and Meeker (1997).

Font Name	Daytime Recognition Distance	Nighttime Recognition Distance
Clear Condensed 100	120 m (394 ft)	86 m (282 ft)
Clear Condensed 112	134 m (440 ft)	105 m (344 ft)
Standard Highway Series D	117 m (384 ft)	86 m (282 ft)
Clear 100	132 m (433 ft)	103 m (338 ft)
Clear 112	144 m (472 ft)	118 m (387 ft)
Standard Highway Series E(M)	137 m (449 ft)	101 m (331 ft)

The results of the word legibility study conducted during the daytime indicated that the microprismatic sheeting produced a 4 percent improvement in legibility distance, compared to the encapsulated lens sheeting. There was no significant interaction between font and material, however. Looking at the effects of font on legibility distance, there was no significant difference in the daytime legibility distances obtained with the Series E(M) font and the Clear 100 and Clear 112 fonts. There was also no significant difference in legibility distance between the Series D font and the Clear 112 and Clear Condensed 112 fonts. However, the all uppercase Series D font showed significantly longer legibility distances than the Clear Condensed 100 font.

At nighttime, there was a significant interaction effect between font and sheeting material, such that the Clear 112 font produced significantly longer legibility distances (22 percent longer) than the Series E(M) font, using the encapsulated lens sheeting. The microprismatic showed the same trend (although not significant), with the Clear 112 font producing 11 percent longer legibility distances than the Series E(M). There were no differences between the all uppercase Series D font and the same-size, mixed-case Clear fonts (i.e., Clear 112 and Clear Condensed 112). However, the Series D font produced significantly longer legibility distances than the Clear Condensed 100 font at night. The legibility distances obtained for the six fonts studied under daytime and nighttime are shown in table 16.

Garvey, Pietrucha, and Meeker (1997) state that guide signs are read using both legibility and recognition criteria, depending on the familiarity of a traveler with the location words used on the signs. A driver who is looking for a particular word on a sign, will be able to read it at a farther distance that a driver who has no idea of what might be on the sign. In the legibility task, the larger letters used with the all-upper series D font produced greater legibility distances than the smaller mixed case Clear 100 Condensed font. But when the mixed-case font was increased to take up the same sign area as the Series D font (Clear Condensed 112), the legibility distances for the mixed-case and upper-case fonts were the same. But in the recognition task, for which Garvey, Pietrucha, and Meeker (1998) state more closely represents real-world behavior, the same-size, mixed-case fonts performed significantly better than the all upper-case Series D font. And, even the mixed-case font that took up less sign space performed as well as the all uppercase, Series D font, in terms of word recognition. The authors explain that upper-case words look like blurry rectangles when viewed from a distance. Mixed-case font, on the other hand, produces words with a recognizable overall shape, due to the ascending and descending elements in each letter. The data from this study indicate that if the size of mixed-case words on a sign is matched to the size of words presented in all uppercase font, the mixed-case font provides equal legibility distance and superior recognition distance.

Table 16. Daytime and Nighttime Legibility Distances for Fonts Studied by Garvey, Pietrucha, and Meeker (1997).

Font Name	Daytime Legibility Distance	Nighttime Legibility Distance
Clear Condensed 100	57 m (187 ft)	45 m (148 ft)
Clear Condensed 112	67 m (220 ft)	59 m (194 ft)
Standard Highway Series D	68 m (223 ft)	63 m (207 ft)
Clear 100	67 m (220 ft)	60 m (197 ft)
Clear 112	70 m (230 ft)	75 m (246 ft)
Standard Highway Series E(M)	68 m (223 ft)	60 m (197 ft)

Next, the MUTCD states that street-name signs should be placed at least on diagonally opposite corners so that they will be on the far right-hand side of the intersection for traffic on the major street. Burnham (1992) noted that signs located over the highway are more likely to be seen before those located on either side of the highway. In this regard, Zwahlen (1989) examined detection distances of objects in the peripheral field versus line-of-sight detection and found that average detection distances decrease considerably as the peripheral visual detection angle increases. Placement of street-name signs overhead places the sign in the driver's forward line of sight, eliminating the need for the driver to take his/her eyes away from the driving scene, and reduces the visual complexity of the sign's surround, but under some sky conditions (e.g., backlit by the sun at dawn and dusk) the sign may be unreadable. Thus, overhead street-name signing should be a supplement to standard roadside signing.

The use of an advance street-name plaque (W16-8) with an advance warning crossroad, side road, or T-intersection sign (W2-1, W2-2, W2-3, and W2-4) provides the benefit of additional decision and maneuver time prior to reaching the intersection. Section 2C.45 of the MUTCD (2000) indicates the use of such supplemental street-name signs on intersection warning signs as an option (e.g., an advance street-name plaque *may* be erected separately or below an intersection-related warning sign). The use of advance street-name plaques on advance warning signs has been successful in Phoenix, AZ (*Rural and Urban Roads*, 1973); the size of the lettering on these signs is 200 mm (8 in). Midblock street-name signing provides the same benefit.

Finally, noting Mace's (1988) conclusions supporting a legibility index as conservative as 0.36 m/mm (30 ft/in) to accommodate older drivers, and the practical limitations of increasing sign panel size, a justification emerges for eliminating the border on street-name signs to permit the use of larger characters. The MUTCD (2000) section 2A.15 states that, "Unless specifically stated otherwise, each sign illustrated herein shall have a border of the same color as the legend, at or just inside the edge" In section 2D.38 (Street Name Signs), the MUTCD states that, "A border, if used, should be the same color as the legend." The border on street-name signing is presumed to enhance the conspicuity of the sign panel at intersections, where visual complexity and driving task demands may be relatively high. However, the aspect of conspicuity at issue here is "search conspicuity" rather than "attention conspicuity;" as demonstrated by Cole and Hughes (1984), a sign is noticed at significantly greater distance when a driver expects its presence and knows where to look for it. This is the case with street-name signing at intersections. Detecting the presence of street-name signs isn't the problem--reading them is. Thus, a strong argument can be made that any marginal reduction in conspicuity that may result from eliminating sign borders will be more than offset by the resultant gains in legibility produced by larger characters in the sign legend.

K. Design Element: One-Way/Wrong-Way Signing

Table 17. Cross-references of related entries for one-way/wrong-way signing.

Applications in Standard Reference Manuals		
MUTCD (2000)	**AASHTO Green Book (1994)**	**Traffic Engineering Handbook (1999)**
Sect. 1A.13, *regulatory signs* & *wrong-way arrows* Sects. 2A.24, 2B.05, 2B.28 through 2B.30, 2B.32, 2B.33 & 2E.50 Figs. 2A-3 through 2A-6 & 2E-31 through 2E-32	Pg. 519, Paras. 4-5 Pg. 726, Para. 4 Pg. 915, Para. 6	Pg. 384, 1st Principle Pg. 424, Para. 1 Pg. 426, Paras. 1-4 Pg. 436, Sect. on *Word and Symbol Markings* Pg. 438, Item 4

Vaswani (1974, 1977) found that approximately half of the incidents that involved wrong-way driving on multilane divided highways without access control occurred at intersections with freeway exits and with secondary roads. These wrong-way movements resulted from left-turning vehicles making a left turn into a lane on the near side of the median, rather than turning around the nose of the median into a lane on the far side. In an analysis of 96 crashes resulting from wrong-way movements on divided highways in Indiana from 1970 through 1972, Scifres and Loutzenheiser (1975) found that wrong-way movements most often occur under conditions of low traffic volume, low visibility, and low lane-use density. In addition, it was reported that

69 percent of the wrong-way drivers were drunk, older (age 65 and older), or fatigued (driving between 12 a.m. and 6 a.m.). A review of the literature by Crowley and Seguin (1986) reported that (1) there are significantly more incidents of wrong-way driving than there are crashes, and (2) drivers older than 60 years of age are overrepresented in wrong-way movements on a per-mile basis.

Further evidence of older driver difficulties likely to result in wrong-way movements was reported by McKnight and Urquijo (1993). These researchers examined 1,000 police forms that documented observations of incompetence when an older driver was either stopped for a violation or involved in a crash. They found that two of the primary behaviors that brought these drivers to the attention of police were driving the wrong way on a one-way street and driving on the wrong side of a two-way street. The drivers' mistakes contributed to many violations (149) but few crashes (29).

The ability to abstract information and make quick decisions are capabilities required to safely perform the driving task. Evidence has been found that older drivers' crashes often occur as the result of overly attending to irrelevant aspects of a driving scene (Planek and Fowler, 1971). Hasher and Zacks (1988) argued that older adults are deficient in inhibitory processes, and as a result, they frequently direct attention to irrelevant information at the expense of relevant information. The selective attention literature generally suggests that for adults of all ages, but particularly for the elderly, the most relevant information must be signaled in a dramatic manner to ensure that it receives a high priority for processing in situations where there is a great deal of complexity. Mace (1988) stated that age differences in glare sensitivity and restricted peripheral vision coupled with the process of selective attention may cause higher conspicuity thresholds for older drivers. Overall, these deficits point to the need for more effective and more conspicuous signs, realized through provision of multiple or advance signs as well as changes in size, luminance, or placement of signs.

The most comprehensive survey of current policies and practices for signing intersections to inform drivers of travel direction and to prevent wrong-way movements was conducted in the 48 contiguous States and in 35 of the largest cities by Crowley and Seguin (1986). They found considerable variability in the location, placement, and types of signs used to prevent wrong-way movements from occurring. The greatest variability in practice was reported in locations where a median divider exists. The study authors reported that median width is a key factor in the number, type, and location of signs to be used. When medians are extremely narrow, there appears to be little confusion that the intersecting roadway is two-way and drivers have less need for special signs to indicate travel direction. Where the median is sufficiently large, the intersection will be generally signed as two separate one-way roadways. A problem in defining what is "wide" and what is "narrow" was shown in the responses from a survey of practitioners across the United States, where there was a significant range in values around the 9 m (30 ft) delineation point specified by the MUTCD (para. 2A-24). The majority of jurisdictions tended to treat wide-median divided highways as if they were two separate intersections for the purpose of direction and turn-prohibition signing. The most commonly reported sign configuration implemented in the jurisdictions that responded to the survey was the MUTCD standard of a pair of ONE WAY signs (R6-1) on the near right-hand corners and far left-hand corners of *each* intersection with the directional roadway. A second pattern reported was a slight variation of the MUTCD standard, where the jurisdictions required a far-right sign (either a ONE WAY or a NO RIGHT TURN symbol sign) at the second intersection. Although many jurisdictions followed the MUTCD specifications for location of signs, many reported that they *replaced* a near-side ONE WAY sign with a NO RIGHT TURN sign (R3-1), even though the MUTCD states that the turn prohibition sign may be used to *supplement* the near-right/far-left pair of ONE WAY signs. The third pattern reported by some jurisdictions was to treat the divided highway, *regardless of median width*, as if it were a single intersection. In this case, a left/median sign for the first one-way roadway and a far-right sign for the second one-way roadway were considered sufficient. Where jurisdictions implement the third pattern, there was more emphasis on the use of the DIVIDED HIGHWAY CROSSING sign (R6-3) to supplement the limited amount of directional information. In one jurisdiction, signing was limited to the use of the DIVIDED HIGHWAY CROSSING sign.

Crowley and Seguin (1986) reported that some jurisdictions recommended the use of optional signs--i.e., DO NOT ENTER (R-5-1), WRONG WAY (R5-9), and KEEP RIGHT (R4-7)--but noted that these signs are not helpful to a motorist making decisions as he/she approaches an intersection; they are detected only when the driver begins a wrong-way movement upon reaching the intersection. In this regard, a number of jurisdictions reported that they required the use of the DIVIDED HIGHWAY CROSSING sign, as it is the only sign available that has a direct impact on the decision process of drivers approaching a divided highway with a median. The MUTCD states that this sign may be used as a supplemental sign on the approach legs of a roadway that intersects with a divided highway. Although this sign was not included in the set of traffic control devices tested by Hulbert and Fowler (1980), these researchers found that where complex driver judgments were required in conjunction with the use and understanding of particular driving situations, larger percentages of drivers failed to correctly respond to the meaning of traffic control devices. The comprehensibility of the DIVIDED HIGHWAY CROSSING sign has not been reliably documented.

Crowley and Seguin (1986) also conducted a laboratory study and a field validation study using subjects in three age groups (younger than age 25, ages 25-54, and age 55 and older) to identify signing practices that best provide information to minimize the possibility of wrong-way turning movements. Subjects were asked to identify driver actions that were either directly or by implication prohibited (by signs, markings, etc.), and to do so as quickly as possible. In the laboratory study, projected scenes of intersections containing a median (divided highway) were associated with higher error rates and longer decision latencies than scenes containing T-intersections and intersections of a two-way street with a one-way street (no median). The untreated intersections, where geometry alone was tested to determine the extent to which it conveyed an intrinsic "one-way" message, resulted in the worst performance; thus, any signing, regardless of the configuration, appears to be superior to no signing. However, even when the standard MUTCD near-right/far-left placement of ONE WAY signs was presented, large numbers of subjects did not recognize that the projected scene was that of a divided highway. Furthermore, the addition of a DIVIDED HIGHWAY CROSSING sign at the near-right corner of the intersection did not significantly reduce the overall error rate. Subjects age 55 and older had fewer correct responses and longer decision latencies than subjects in the two younger age groups. Field study results showed the following: (1) unsignalized divided highways resulted in more extreme steering patterns than signalized divided highways, at both of the one-way locations; (2) the use of ONE WAY signs in the left/median and far-right locations for medians as narrow as 6 m (20 ft) and as wide as 12.8 m (42 ft) showed superior performance to the single left/median ONE WAY sign; and (3) at undivided intersections of a two-way street with a one-way street, the most extreme variation in steering position was shown for the untreated intersections, suggesting that any signing treatment is better than none.

Crowley and Seguin (1986) noted that because there are intersections with specific physical factors that make the basic near-right/far-left rule inappropriate, the following text should be added to the MUTCD (1988) in section 2B-29 (section 2B.32 of the MUTCD, 2000) to bring the MUTCD and actual practice more in agreement and to reflect the actual manner in which the practitioner must respond to the problem of signing to prevent wrong-way traffic movements while providing positive guidance to drivers: "However, if an engineering study demonstrates the specified placements to be inappropriate due to factors such as sight distance restrictions, approach roadway grade and/or alignment, complex background, etc., one-way signs should be placed so as to provide the best possible guidance for the driver." In addition, a revision to MUTCD (1988) section 2A-31 was proposed (section 2A.24 of the MUTCD, 2000) which states that for medians of 9 m (30 ft) and under, both the left/median and far-right locations should be implemented when a divided highway justifies any form of one-way signing (see figure 12). DIVIDED HIGHWAY CROSSING, DO NOT ENTER, and WRONG WAY signs are optional, depending on the specific problem at a narrow median intersection. The authors note, however, that when a median is very narrow, one-way signing is usually unnecessary.

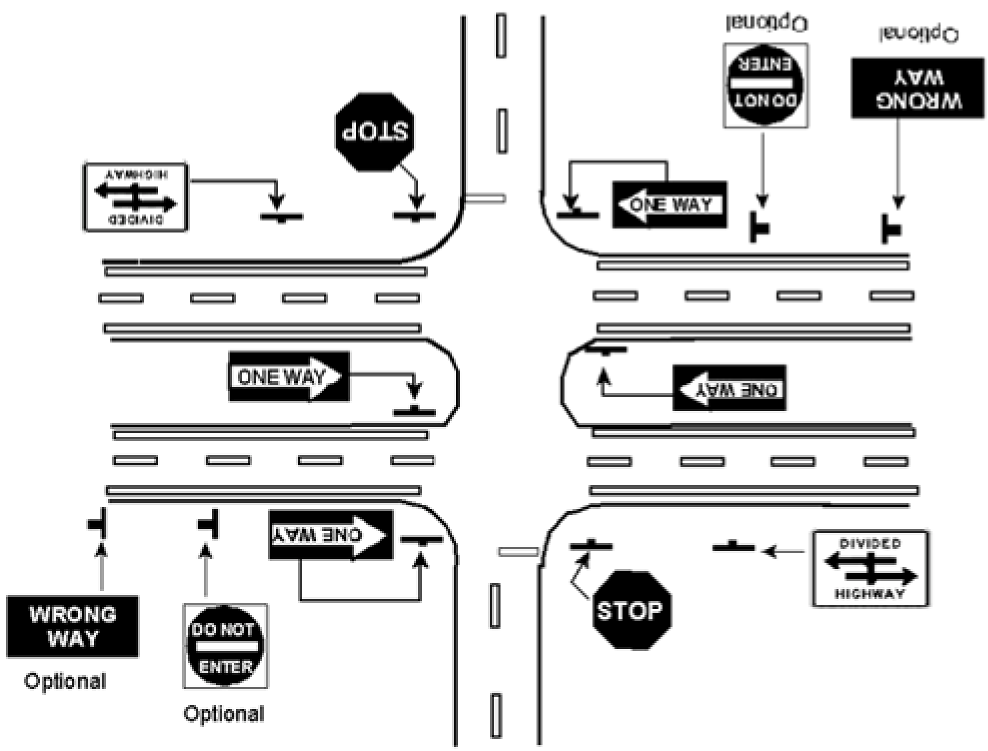

Figure 12. Recommended signing configuration for divided highway crossings for medians less than or equal to 9 m (30 ft), based on evidence provided by Crowley and Sequin, 1986.

For medians greater than 9 m (30 ft), Crowley and Seguin (1986) suggested the use of ONE WAY signs posted at each of the following locations, for each direction of traffic: near right, median left, and far right. WRONG WAY and DO NOT ENTER signs are again optional. The resulting configuration is consistent with that shown earlier in Recommendation 4 of Design Element E. Because of the large observation and entrance angles for the ONE WAY, KEEP RIGHT, DO NOT ENTER, and WRONG WAY signs, signs using sheeting that provides for high retroreflectivity overall, and particularly at wide observation angles and extended entrance angles are required; otherwise the signs will be virtually invisible.

For T-intersections, Crowley and Seguin (1986) recommended that near-right side ONE WAY signs and far side ONE WAY signs be located so that drivers are most likely to see them *before* they begin to make a wrong-way movement. The optimal placement for the far side sign would be opposite the extended centerline of the approach leg as shown in MUTCD (1988) figure 2-4 (figure 2A-6, for the MUTCD, 2000). However, where a study indicates that the far-side centerline location is not appropriate at a particular intersection because of blockage, distracting far-side land use, excessively wide approach leg, etc., these authors suggested that the best alternate location is the far left-hand corner for one-way traffic moving from left to right, and the far right-hand corner for traffic moving from right to left (see figure 13).

For four-legged intersections (i.e., the intersection of a one-way street with a two-way street), the near-right/far-left locations were recommended by Crowley and Seguin (1986) regardless of whether there is left-to-right or right-to-left traffic. An additional ONE WAY sign

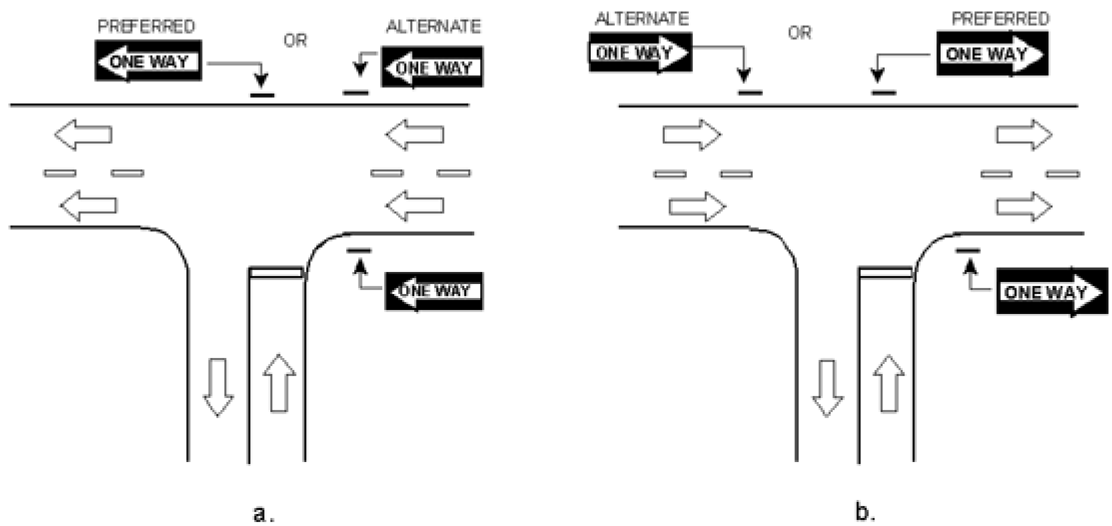

Figure 13. Recommended locations of ONE WAY signs for T-type intersections.
Source: Crowley and Seguin, 1986.

located on the far-right side may be necessary in certain locations where approach grade and angle may direct the driver's field of view away from the "normal" sign locations (see figure 14).

Finally, as noted in the "Rationale and Supporting Evidence" for Design Element E, the potential for wrong-way movements at intersections with channelized (positive) offset left-turn lanes (within a raised median) increases for the driver turning left from the minor road onto the major road, who must correctly identify the proper median opening into which he/she should turn. The following countermeasures were recommended at intersections with a divided median on the receiving leg, where the left-turn lane treatment results in channelized offset left-turn lanes (e.g., a parallel or tapered left-turn lane between two medians); these countermeasures are intended to reduce the potential for wrong-way maneuvers by drivers turning left from the stop-controlled minor roadway:

- Proper signing (DIVIDED HIGHWAY CROSSING signs, and proper positioning of WRONG WAY, DO NOT ENTER, and ONE WAY signing at the intersection) must be implemented.

- The channelized left-turn lanes should contain white lane-use arrow pavement markings (left-turn only).
- Pavement markings that scribe a path through the turn are recommended to reduce the likelihood for the wrong-way movement.
- Placement of 7.1-m- (23.5-ft-) long wrong-way arrows in the through lanes is recommended, as specified in the MUTCD (2000) for wrong-way traffic control for locations determined to have a special need, sections 2A.24 and 2E.50. Wrong-way arrows have been shown to reduce the frequency of wrong-way movements at freeway interchanges (Parsonson and Marks, 1979).
- Indistinct medians are considered to be design elements that tend to reduce a driver's ability to see and understand the overall physical and operational features of an intersection, increasing the frequency of wrong-way movements (Scifres and Loutzenheiser, 1975). Delineation of the median noses using reflectorized treatments will increase their visibility and should improve driver understanding of the intersection design and function.

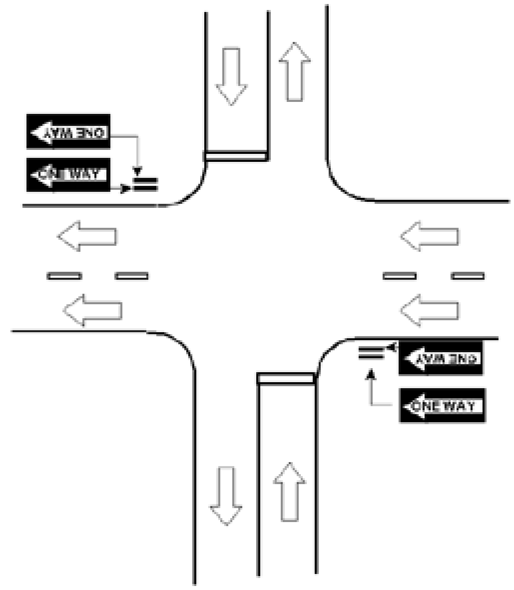

Figure 14. Recommended locations of ONE WAY signs for intersection of one-way and two-way street. Source: Crowley and Seguin, 1986.

The recommended placement of these traffic control devices was illustrated in Recommendation 4 of Design Element E.

L. Design Element: Stop- and Yield-Controlled Intersection Signing

Table 18. Cross-references of related entries for stop- and yield-controlled intersection signing.

Applications in Standard Reference Manuals			
MUTCD (2000)	**AASHTO Green Book (1994)**	**NCHRP 279 Intersection Channelization Design Guide (1985)**	**Traffic Engineering Handbook (1999)**
Sect. 1A.13, *regulatory signs* Tables 2B-1 & 2C-4 Sects. 2B.03 through 2B.10, 3B.15 & 2C.26	Pgs. 117-125, Sect. on *Stopping Sight Distance* Pg. 698, Fig. IX-32B Pgs. 700-703, Sects. on *Case II--Yield Control for Minor Roads* & *Case III--Stop Control on Minor Roads* Pg. 739, Para. 3 Pg. 919, Sect. on *At-grade terminals* Pg. 939, Para. 2	Pg. 9, Figs. 2-5 & 2-7 Pg. 10, Table 2-4, 4th bullet Pg. 21, Fig. 3-1	Pgs. 235-237, Sects. on *Yield Control* & *Stop Control* Pg. 419, Sect. on *Size* Pgs. 421-423, Sect. on *Types of Retroreflective Sheeting Material* Pg. 426, Last Para. Pg. 427, Paras. 1-3 Pg. 443, Sect. on *Rumble Strips and Speed Humps* Pgs. 444-445, Sects. on *STOP Sign Warrants, Multiway STOP Warrants,* & *YIELD Sign Warrants*

Drivers approaching a nonsignalized intersection must be able to detect the presence of the intersection and then detect, recognize, and respond to the intersection traffic control devices present at the intersection. Next, drivers must detect potential conflict vehicles, pedestrian crosswalk locations, and pedestrians at or near the intersection. Proper allocation of attention has become more difficult, as drivers are overloaded with more traffic, more signs, and more complex roadway configurations and traffic patterns, as well as more complex displays and controls in newer vehicles (Dewar, 1992). The presence of large commercial signs near intersections has been associated with a significant increase in crashes at stop-controlled intersections (Holahan, 1977).

Age-related deficits in vision and attention are key to developing recommendations for improved stop control and yield control at intersections. Researchers examining the State crash records of 53 older drivers found that those with restrictions in their "useful field of view," a measure of selective attention and speed of visual processing, had 15 times more intersection crashes than those with normal visual attention (Owsley, Ball, Sloane, Roenker, and Bruni, 1991). A follow-up study with a sample of 300 drivers demonstrated that visual attention deficits could account for up to 30 percent of the variance in intersection crash experience (Ball, Owsley, Sloane, Roenker, and Bruni, 1993). Additional relevant findings may be cited from a simulator study of peripheral visual field loss and driving impairment which also examined the actual driving records of the study participants (Szlyk, Severing, and Fishman, 1991). It was found that visual function factors, including acuity as well as visual field measures, could account for 26 percent of the variance in real-world crashes. Also, greater visual field loss was associated in the simulator data with greater distance traveled ("reaction distance") before responding to a peripheral stimulus (e.g., a STOP sign).

A considerable body of evidence exists documenting the difficulties of older driver populations in negotiating stop-controlled intersections. Specifically, analyses of crash and violation types at these sites highlight the older driver's difficulty in detecting, comprehending, and responding to signs within an appropriate timeframe for the safe completion of intersection maneuvers.

Statistics on Iowa fatal crashes show that during 1986-1990, running STOP signs was a contributing circumstance in 297 fatal crashes which killed 352 people; drivers age 65 and older accounted for 28 percent of the fatal crashes, and drivers younger than age 25 were involved in 27 percent of the fatal crashes (Iowa Department of Transportation, 1991). Stamatiadis, Taylor, and McKelvey (1991) found that at stop-controlled urban intersections, the percentage of drivers age 75 and older involved in right-angle crashes was more than double that of urban signalized intersections. Malfetti and Winter (1987), reporting on the unsafe driving performance of drivers age 55 and older, noted that older drivers frequently failed to respond properly or respond at all to road signs and signals; descriptions of their behavior included running red lights or STOP signs and rolling through STOP signs. Some older persons' behavior at STOP signs and signals seemed to indicate that they did not understand why they needed to wait when no other traffic was coming. Brainan (1980) used in-car observation to gain firsthand knowledge and insight into older people's driving behavior. Drivers in the 70 and older age group showed difficulty at two of the STOP signs on the test route; their errors were in failing to make complete stops, poor vehicle positioning at STOP signs, and jerky and abrupt stops. Campbell, Wolfe, Blower, Waller, Massie, and Ridella (1990), looking at police reports of crossing crashes at nonsignalized intersections, found that older drivers often stopped and then pulled out in front of oncoming traffic, whereas younger drivers more often failed to stop at all. Further evidence of unsafe behaviors by older drivers was provided in a study by McKnight and Urquijo (1993). Their data consisted of 1,000 police referral forms from the motor vehicle departments of California, Maryland, Massachusetts, Michigan, and Oregon; the forms included observations of incompetent behavior exhibited by older drivers who were stopped for a violation by law enforcement personnel or were involved in a crash. The specific behaviors contributing to the contact between the older driver and the police officer included failing to yield right-of-way or come to a complete stop at a STOP sign, and failing to stop or yield to other traffic; taken together, these behaviors contributed to significant numbers of crashes (74) and violations (114).

Data from 124,000 two-vehicle crashes (54,000 crashes at signalized intersections and 70,000 crashes at nonsignalized intersections) showed that drivers younger than age 25 and older than age 65 were overinvolved in crashes at both types of intersections (Stamatiadis et al. 1991). However, the overinvolvement of older drivers in *nonsignalized* intersection crashes was more pronounced than it was for signalized intersection crashes. Although the total number of crashes was reduced at nonsignalized intersections that contained signs when compared with unsigned intersections, the crash involvement ratios of older drivers were higher at signed intersections than at unsigned intersections. At nonsignalized intersections, the highest percentage of fatalities were the result of right-angle collisions (25 percent). In terms of the frequency of injury at nonsignalized intersections, rear-end crashes were the most frequent cause (35 percent), followed by right-angle crashes (18 percent), other-angle crashes (10 percent), and head-on/left-turn crashes (8 percent). The leading violation types for all older drivers in descending order were failure to yield right-of-way, following too closely, improper lane usage, and improper turning. At nonsignalized intersections, older drivers showed the highest crash frequency on major streets with two lanes in both directions (a condition most frequently associated with high-speed, low-volume rural roads), followed by roads with four lanes, and those with five lanes in both directions. These configurations were most often associated with low-speed, high-volume urban locations, where intersection negotiation involves more complex decisions involving more conflict vehicles and more visually distracting conditions.

Cooper (1990) utilized a database of all 1986 police-attended crashes in British Columbia, in an effort to compare the crash characteristics of older drivers with those of their younger counterparts. While 66.5 percent of crashes involving drivers ages 36-50 occurred at intersections, the percentage increased to 69.2 percent, 70.7 percent, and 76.0 percent for drivers ages 55-64, 65-74, and 75 and older, respectively. Overall, the two oldest groups identified in this analysis were significantly more crash involved at

STOP/YIELD sign locations and less involved at either uncontrolled or signal-regulated locations. In follow-on questionnaires administered to a sample of drivers in each age group studied, intersection negotiation was mentioned by the older drivers as second in difficulty to problems changing lanes. About 20 percent of the older drivers mentioned not stopping properly at STOP signs. Vehicle maneuvering prior to the crash was a key variable for drivers over age 65, and in particular, for left turns at uncontrolled or STOP/YIELD sign-controlled intersections. Drivers ages 36-50 experienced only 10.9 percent of their crashes while turning left at this type of intersection, compared with 13.0, 15.4, and 19.5 percent of drivers ages 55-64, 65-74, and 75 and older, respectively.

Council and Zegeer (1992) conducted an analysis of intersection crashes occurring in Minnesota and Illinois for the time period of 1985-1987 to highlight crash types, situations, and causes of crashes, in an effort to increase the knowledge of how older drivers react at intersections. For all the analyses, comparisons were made between a "young-old" group (ages 65-74), an "old-old" group (age 75 or older), and a "middle-aged" comparison group (ages 30-50). Their findings regarding driver age differences in collision types, pre-crash maneuvers, and contributing factors are described below.

With respect to collision type at stop-controlled intersections, analysis of the data showed little difference in the proportion of crashes involving left-turning vehicles at either urban or rural locations when the older groups were compared with the middle-aged group. There was, however, a significant overinvolvement for both groups of older drivers in right-angle collisions, both in urban and in rural locations. At urban intersections, right-angle collisions accounted for 56.1 percent of the middle-aged driver crashes, compared with 64.7 percent of the young-old, and 68.3 percent of the old-old driver crashes. These percentages increase for all groups at rural intersections--61.3, 68.6, and 71.2 percent, respectively for middle-aged drivers, young-old drivers, and old-old drivers. Data for yield-controlled intersections showed older drivers overcontributing to left-turn collisions in urban areas and to angle collisions in both urban and rural areas.

Regarding pre-crash maneuvers at stop-controlled intersections, for both rural and urban locations, right-angle collisions were the most frequent collisions, and middle-aged drivers were more likely to be traveling straight or slowing/stopping than the two older groups. The older drivers were more likely to be turning left or starting from a stop than their younger counterparts. The pattern is similar for left-turning crashes. For rear-end collisions, the old-old drivers were more likely to be going straight (thus being the striking vehicle), and the middle-aged and young-old drivers were more likely to be stopped or slowing. For the few right-turning collisions at urban stop-controlled intersections, the middle-aged drivers were going straight and the old-old drivers were more likely to be turning left or right or starting from a stop. Rural stop-controlled locations showed the same patterns of precrash maneuvers among the three age groups.

Finally, breakdowns of contributing factors for the urban and rural stop-controlled intersections showed that the middle-aged drivers exhibited a higher proportion of no improper driving behavior, while the young-old and old-old drivers were more often cited for failure-to-yield, disregarding the STOP sign, and driver inattention. When starting from a stop, however, the old-old drivers had a *lower* probability of being cited for improper driving. When cited, the old-old group was more likely to have disregarded the STOP sign than the other two driver groups. The young-old drivers as well as the old-old drivers more frequently failed to yield than the middle-aged drivers.

For left turns, the middle-aged drivers again were more frequently found to have exhibited "no improper driving." The two older driver groups were most frequently cited with failure-to-yield. There was no difference in the number of drivers in each age group who disregarded the STOP sign. For going-straight situations, the middle-aged driver was found to have exhibited no improper driving behavior twice as often as the young-old driver and almost three times as often as the old-old driver. Failing to yield, disregarding the STOP sign, and inattention were most often cited as the contributing factor for the two older groups.

Signing countermeasures to improve safe operation by older drivers at stop- and yield-controlled intersections follow.

Greene, Koppa, Rodriguez, and Wright (1996) noted that the MUTCD provides for the possibility of enlarging STOP signs where greater emphasis or visibility is required. They proposed an enlargement from the current 750 x 750 mm (30 x 30 in) to 900 x 900 mm (36 x 36 in) at well-traveled intersections or at intersections of small country lanes with State highways. This would also be appropriate at intersections where there is a high incidence of STOP-sign running. Further, Swanson, Dewar, and Kline (1994) reported that older drivers participating in focus group discussions in Calgary, Canada, Boise, Idaho, and San Antonio, Texas indicated a need for bigger and brighter STOP signs.

Mace and Pollack (1983) noted that conspicuity is not an observable characteristic of a sign but a construct which relates measures of perceptual performance with measures of background, motivation, and driver uncertainty. In this regard, conspicuity may be aided by multiple treatments or advance signing as well as changes in size, contrast, and placement. They noted that STOP signs following a STOP AHEAD (W3-1a) sign are more conspicuous not only to older drivers but to everyone, because expectancy has been increased.

The need for appropriate levels of brightness to ensure conspicuity and timely detection by drivers of highway signs, including STOP and YIELD signs, was addressed in FHWA-sponsored research to establish *minimum* retroreflectivity requirements for these devices (minimum maintained levels, as opposed to new or in-service levels). Mace developed a model to derive the retroreflectivity levels necessary for adequate visibility distance, taking into account driver age and visual performance level, as well as the driver's response requirements (action versus no action) to the information presented on a given sign when encountered in a given situation (city, highway) with an assumed operating speed (ranging from 16 km/h [10 mi/h] to 104 km/h [65 mi/h]), for signs of varying size and placement (shoulder, overhead). This work is reported by Ziskind, Mace, Staplin, Sim, and Lococo (1991), who conducted laboratory and controlled field studies using 200 younger and older drivers (ages 16 to 70+) to determine the minimum visibility requirements for traffic control devices. Taking speed and sign application into account, the recommended (minimum maintained, below which the sign should be replaced) retroreflectivity for STOP signs resulting from this research ranged between 10 cd/lux/ m^2 up to 24 cd/lux/ m^2 for the sign background (red) area, with significantly higher values for the sign legend. For the YIELD sign, the recommended minimum maintained levels ranged between 24 and 39 cd/lux/ m^2. These units, in cd/lux/m^2, or coefficient of retroreflection (R_A) express the efficiency with which the material is able to return incident light at a given geometry between the sign, the vehicle, and the driver. A retroreflectometer is used to obtain these data in the field; reflectivity of a material is measured at specific angles. The observation angle is the angle between the headlamps, the sign, and the driver's eye. The R_A measurements provided by FHWA are all measured at a 0.2 degree observation angle, which corresponds roughly to a viewing distance of 213 m (700 ft), for a right shoulder-mounted sign on a straight road viewed from a passenger sedan. This is important, because in general, as a vehicle approaches a sign, the observation angle becomes larger, reaching 1.0 degrees at 91 m (300 ft), which is roughly legibility distance. Knowing the R_A of a material at 0.2 degrees does not automatically predict its reflectivity at a closer distance (larger observational angle). Because both the STOP and YIELD signs are so extensively overlearned by drivers, their comprehension is believed to be associated with the icon, i.e., their unique shape and coloration. Thus, the brightness of the sign's background area is most critical, because these devices will typically be recognized and understood as soon as they are detected (the conspicuity distance), rather than closer in (legibility distance).

Mercier, Goodspeed, Simmons, and Paniati (1995) conducted a laboratory study using younger and older drivers to measure the minimum luminance thresholds for traffic sign legibility, to accommodate varying percentages of the driving population. The purpose of the study was to evaluate the proposed minimum retroreflectivity values derived using CARTS (Computer Analysis of the Retroreflectance of Traffic Signs) that uses a mathematical model to study the relationships between driver variables, vehicle variables, sign variables, and roadway variables (Paniati and Mace, 1993). This model uses MRVD (Minimum Required Visibility Distance), which is the shortest distance at which a sign must be visible to enable a driver to respond safely and appropriately, and includes the distance required for a driver to detect the sign, recognize the message, decide on a proper action, and make the appropriate maneuver before the sign moves out of the driver's view. Paniati and Mace's minimum inservice values (below which sign replacement is indicated) were reported to accommodate an unknown level between 75 to 85 percent of all drivers (see table 19).

The subjects in the Mercier et al. (1995) study included 10 drivers ages 16 to 34; 10 drivers ages 35 to 44; 10 drivers ages 45 to 54; 10 drivers ages 55 to 64; 13 drivers ages 65 to 74; and 10 drivers age 75 or older. All subjects had a visual acuity of at least 20/40. Subjects viewed 25 scaled signs at two distances to simulate minimum required visibility distances (MRVD) traveling at 48 km/h (30 mi/h) and 88 km/h (55 mi/h). Among the signs tested were white-on-red regulatory signs. Illumination levels were manipulated using 20 neutral density filters ranging from 0.02 to 3.0. Type I engineering grade sheeting was used for all signs.

Retroreflectance values were calculated based on the luminance levels needed to accommodate 67, 85, and 95 percent of the population of U.S. drivers. Mercier et al. (1995) concluded that the values recommended by Paniati and Mace (1993), reproduced in table 19 for the white on red signs, are sufficient to accommodate a high percentage of drivers, with the exception of a few signs, which includes the YIELD sign. The 95th percentile driver could not be accommodated by the minimum retroreflectivity suggested for the YIELD sign measuring 76 cm (30 in), for MRVD at both 48 and 88 km/h. The authors point out that increasing brightness for this sign does not increase legibility for older drivers; instead, a redesign of the sign or an enlargement would be needed to enable older drivers to resolve the level of detail required for recognition.

Table 19. Minimum (maintained) retroreflectivity guidelines for white on red signs specified by Paniati and Mace (1993) to accommodate 75 to 85 percent of all drivers.

Sign Size	Speed	Minimum Retroreflectivity cd/lux/m^2

		(Paniati and Mace)
76 cm (30 in)	72 km/h (45 mi/h)	70 (white) 14 (red)
76 cm (30 in)	64 km/h (40 mi/h)	40 (white) 8 (red)
91 cm (36 in)	72 km/h (45 mi/h)	60 (white) 12 (red)
91 cm (36 in)	64 km/h (40 mi/h)	35 (white) 7 (red)
122 cm (48 in)	72 km/h (45 mi/h)	50 (white) 10 (red)
122 cm (48 in)	64 km/h (40 mi/h)	30 (white) 6 (red)

Next, there has been increasing interest in the use of durable fluorescent sheeting for highway signs, because of its increased conspicuity over standard highway sign sheeting, under daytime conditions. Highway signs with fluorescent sheeting have been found to be more conspicuous and can be detected at a further distance than signs with standard sheeting of the same color. In addition, the color of fluorescent signs is more frequently recognized correctly at farther distances than standard sheeting of the same color (Jenssen, Moen, Brekke, Augdal, and Sjøhaug, 1996; Burns and Pavelka, 1995). Of particular interest, however, are findings reported by Burns and Pavelka (1995) for a field study conducted at dusk (15 min after sunset), without the use of vehicle headlights. In this study, 14 drivers ages 19 to 57 (median age = 40 years) viewed signs with fluorescent red sheeting and signs with standard red sheeting at a distance of 30 m (98 ft). The signs with fluorescent red sheeting were detected by 90 percent of the participants; only 23 percent were able to detect the standard red signs. In terms of correct color recognition, 49 percent were able to correctly recognize the color of the fluorescent red signs at dusk from a distance of 30 m, compared to 12 percent who correctly identified the standard red signs as red. Luminance measurements of the targets and the background were taken for these north-facing signs at dusk, so that luminance contrast ratios could be calculated. The luminance contrast ratio (L_t-L_b/L_b, or, the luminance of the target minus the luminance of the background, divided by the luminance of the background) for the fluorescent red signs was 0.7, and for the standard red signs, the luminance contrast ratio was 0.3. The results of this study suggest that the use of fluorescent red sheeting on STOP signs would serve to increase their conspicuity both under daytime and low luminance conditions, and would be of particular benefit to older drivers, who suffer from decreases in contrast sensitivity and have greater difficulty quickly isolating and attending to the most relevant targets in a cluttered visual background. When additional studies quantify the performance gains for older road users, recommendations for relatively widespread use of fluorescent sheeting keyed to specific characteristics of stop- and yield-controlled intersections are likely to emerge. Present recommendations for applications of fluorescent sheeting are limited to the special cases of controlling prohibited movements on freeway ramps (see Chapter II) and for passive control systems at highway-rail grade crossings (see Chapter V).

A two-way stop requires a driver to cross traffic streams from either direction; this poses a potential risk, because cross traffic may be proceeding rapidly and drivers may be less prepared to accommodate to errors made by crossing or turning drivers. Most critically, drivers proceeding straight through the intersection must be aware of the fact that the cross-street traffic does not stop, and that they must yield to cross-street vehicles from each direction before proceeding through the intersection. Older drivers are disproportionately penalized by the late realization of this operating condition, due to the various sources of response slowing noted earlier. Studies of cross-traffic signing to address this problem have shown qualified but promising results in a number of jurisdictions (Gattis, 1996). Although findings indicate that conversion of two-way to four-way stop operations may be more effective in reducing intersection crashes than the use of cross-traffic signing, there are obvious tradeoffs for capacity from this strategy. However, data from crash analyses in Arkansas, Oregon, and Florida reported by Gattis (1996) showed significant reductions in right-angle crashes after cross-traffic signing was installed at problem intersections. Until recently, there was no standard sign design to convey this message; Ligon, Carter, and McGee (1985) identified a number of alternate wordings used in different States. In addition, a warrant for use of a cross-traffic sign applied in the State of Illinois may be reviewed in the Gattis (1996) article. The MUTCD (2000) indicates in section 2C.27 that a CROSS TRAFFIC DOES NOT STOP plaque (W4-4P) may be used to supplement STOP signs on approaches to 2-way, stop-controlled intersections where road users frequently misinterpret the intersection as a 4-way or all-way stop intersection.

Picha, Schuckel, Parham, and Mai (1996) conducted a survey of 2,129 drivers in five States (CA, MN, MS, PA, and TX) to evaluate driver understanding of right-of-way conditions and preference for supplemental signs at two-way, stop-controlled intersections. The majority of the respondents (59 percent) were between ages 25 and 54, however, 22 percent were age 65 or older. The mail survey presented nine supplemental sign designs (three word messages, three symbol messages, and three word-plus-symbol messages), and respondents were asked to choose the preferred sign in each category that best conveyed the right of way conditions at a two-way, stop-controlled intersection, and then to choose the most preferred design of the three. The sign most often preferred (by 84 percent of the sample) was the CROSS TRAFFIC DOES NOT STOP word message with a horizontal double-headed arrow symbol. When asked whether a supplemental sign was needed at all two-way, stop-controlled intersections to tell drivers who has the right-of-way (a diagram was provided with the question), 44 percent of the drivers responded "yes," 50 percent "no," and 6 percent "not sure." Picha et al. (1996) provided a list of conditions that may lead a driver to misinterpret an intersection to be all-way stop controlled, which would justify a supplemental sign treatment. In addition to intersections converted from four-way to two-way stop control, these include:

- The intersection of two single-jurisdictional roadways (e.g., two state-maintained roadways) in a rural or isolated area.
- Intersections with similar average daily traffic (ADT) volumes on all approaches, but less than the minimum volumes that would warrant the installation of a traffic signal. Typical volumes ranging from 5,000 to 10,000 ADT will not likely meet signal warrants, but could justify a supplemental treatment.
- Intersections with a high conflict frequency and rate, i.e., 20 to 25 conflicts per day (all conflicts combined) or a rate of at least 4 conflicts per 1,000 entering vehicles.
- Intersections with a right-angle crash frequency in the range of three to five (or more) per year. Such a condition may not necessarily meet traffic signal warrants.
- A system of roadway intersections (at-grade) that are not consistent with respect to traffic control schemes.
- Intersections with similar high speeds (i.e., greater than 80 km/h [50 mi/h]) on all approaches.
- Intersections with similar cross-sectional elements (number and width of lanes, shoulders, grades, drainage) on all approaches.

The issue of driver expectancy, a key predictor of performance for older motorists, was addressed in a study by Agent (1979) to determine what treatments would make drivers more aware of a stop-ahead situation. Agent concluded that at rural sites, transverse pavement striping should be applied approximately 366 m (1,200 ft) in advance of the STOP sign to significantly reduce approach speeds. Later research (Agent, 1988) recommended the following operational improvements at intersections controlled by STOP signs: (1) installing additional advance warning signs; (2) modifying warning signs to provide additional notice; (3) adding stop lines to inform motorists of the proper location to stop, to obtain the maximum available sight distance; (4) installing rumble strips, transverse stripes, or post delineators on the stop approach to warn drivers that they would be required to stop; and (5) installing beacons. Although Agent emphasized that beacons do not eliminate the problem of drivers who disregard the STOP sign, flashing beacons used in conjunction with STOP signs at isolated intersections or intersections with restricted sight distance have been consistently shown to be effective in decreasing crashes by increasing driver awareness and decreasing approach speeds (California Department of Public Works, 1967; Cribbins and Walton, 1970; Goldblatt, 1977; King, Abramson, Cohen, and Wilkinson, 1978; Lyles, 1980).

With regard to the crash reduction effectiveness of rumble strips placed on intersection approaches, Harwood (1993) reported that rumble strips can provide a reduction of at least 50 percent in the types of crashes most susceptible to correction, including crashes involving running through a STOP sign. They can also be expected to reduce vehicle speed on intersection approaches and to increase driver compliance with STOP signs. In an evaluation conducted by the Virginia Department of Highways and Transportation (1981a) where rumble strips were installed at stop-controlled intersections, the total crash frequency was reduced by 37 percent, fatal crashes were reduced by 93 percent, injury crashes were reduced by 37 percent, and property-damage-only crashes were reduced by 25 percent. In this study, 39 of the 141 crashes in the before period were classified as being types susceptible to correction by rumble strip installation, particularly rear-end crashes and ran-STOP-sign crashes. The crash rate for these crash types was reduced by 89 percent. Carstens and Woo (1982) found that primary highway intersections where rumble strips were installed experienced a statistically significant reduction in the crash rate in the first year or two following their installation, both at four-way and T-intersections. The crash rate at the 21 study intersections decreased by 51 percent for total crashes and by 38 percent for ran-STOP-sign crashes. Carstens and Woo found no statistically significant change in crash rate at 88 intersections on secondary roads where rumble strips were installed. They concluded that rumble strips are more effective at primary highway intersections than secondary road intersections for the following reasons: (1) primary highways serve a higher proportion of drivers who are unfamiliar with the highway; (2) trips tend to be longer on primary highways so that fatigue and the monotony of driving may play a more important role than on secondary roads; (3) traffic volumes are higher on primary highways, so the number of potential conflicts is greater; and (4) the geometric layout of primary highway intersections is often more complex than that of secondary road intersections. These researchers also found that rumble strips may be more effective in reducing nighttime crashes at unlighted intersections than at lighted intersections. Harwood (1993) reported that several highway agencies commented that it was important to avoid the temptation to use rumble strips where they are not needed; if every intersection had rumble strips on its approach, rumble strips would soon lose their ability to focus the attention of the motorist on an unexpected hazard.

Before concluding this discussion, certain aspects of YIELD sign operations deserve mention. A YIELD sign facilitates traffic flow by preventing unnecessary stops and allowing drivers to enter the traffic flow with minimum disruption of through traffic. Most YIELD signs are posted where right-turning drivers can approach the cross street at an oblique angle. Such configurations benefit elderly drivers in carrying out the turning maneuver by avoiding the tight radii that characterize right-angle turns. However, in several respects, intersections regulated by YIELD signs place greater demands upon drivers than those employing other controls, in terms of gap selection, difficulty with head turning, lanekeeping, and maintaining or adjusting vehicle speed. The angle of approach to the street or highway being entered ranges from the near perpendicular to the near parallel. The closer the angle is to the parallel, the further the driver must turn his/her head to detect and to judge the speed and distance of vehicles on the road to be entered. Many elderly drivers are unable to turn their heads far enough to get a good look at approaching traffic, while the need to share attention with the road ahead necessarily limits the gap search to 1 or 2 s. Some drivers are reduced to attempting to judge distance and gaps by means of the outside mirror. The inability to judge gaps in this manner often results in the driver reaching the end of the access lane without having identified an appropriate gap. The driver in this situation comes to a complete stop and then must enter the cross street by accelerating from a stopped position. The difficulty in judging gaps may lead to aborted attempts to enter the roadway, leaving the older driver vulnerable to following drivers who direct their attention upstream and fail to notice that a vehicle has stopped in front of them. The need to share attention between two widely separated points results in eyes being off the intended path for lengthy periods. The diversion of attention, along with movement of the upper torso, hampers the older driver's ability to maintain directional control.

McGee and Blankenship (1989) report that intersections converted from stop to yield control are likely to experience an increase in crashes, especially at higher traffic volumes, at the rate of one additional crash every 2 years. In addition, converted yield-controlled intersections have a higher crash rate than established yield-controlled intersections. They note that while yield control has been found to be as safe as stop control at very low volumes, the safety impacts are not well established for higher volume levels. Agent and Deen (1975) reported that rural road crash types at yield-controlled intersections are different from those at stop-controlled intersections. At YIELD signs, more than half of the crashes were rear-end collisions, while more than half of the crashes at STOP signs were angle collisions.

M. Design Element: Devices for Lane Assignment on Intersection Approach

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Table 20. Cross-references of related entries for devices for lane assignment on intersection approach.

Applications in Standard Reference Manuals			
MUTCD (2000)	**AASHTO Green Book (1994)**	**NCHRP 279Intersection Channelization Design Guide (1985)**	**Traffic Engineering Handbook (1999)**
Sect. 1A.13, *approach* Table 2B-1 Sects. 1A.14 & 2B.19 through 2B.48 Fig. 2B-1 Sects. 3A.01, 3A.02, 3A.06, & 3B.05 Figs. 3B-11 Sect. 3B.19 Figs. 3B-19	Pgs. 431-432, Sect. on *Width of Roadway* Pg. 474, Para. 1 Pg. 517, Para. 5 Pgs. 629-641, Sects. on *Three-Leg Intersections*, *Channelized Three-Leg Intersections*, *Four-Leg Intersections*, & *Channelized Four-Leg Intersections* Pg. 740, Paras. 4-5 Pg. 741, Para. 2 through Table IX-15 on pg. 743 Pgs. 744-747, Figs. IX-54 through IX-57 Pgs. 749-751, Sect. on *Speed-Change Lanes at Intersect-ions* Pgs. 778-792, Sects. on *Continuous Left-Turn Lanes (Two-Way)*, *Auxiliary Lanes*, *Simultaneous Left Turns*, *Intersection Design Elements with*	Pg. 1, Item 2, 3rd bullet Pg. 19, Middle fig. Pg. 21, 2nd col., item 1 Pg. 24, Para. 1 & top fig. Pg. 32, Bottom fig. Pg. 34, Para. 1 & two figs. Pg. 35, Top right fig. Pg. 36, Para. 1 & top & bottom figs. Pg. 37, Para. 2 & top two figs. Pgs. 47-48, Sect. on *Warrants/Guide-lines For Use of Left-Turn Lanes* Pg. 51, Fig. 4-12 Pg. 57, Sects. on *Double Left-Turn Lanes--Guidelines for Use* & *Guidelines for Implementation of*	Pg. 241, Sects. on *Pavement markings, Lane-use control signs,* & *Multiple turn lanes* Pg. 384, 2nd & 7th Principles Pgs. 429-430, Sect. on *Overhead Signs* Pg. 434, Sect. on *Transverse Markings* Pg. 454, 5th bullet Pgs. 522-524, Sect. on *Lane-Use Control Signals*

through 3B-22	*Frontage Roads*, & *Bicycles at Intersections*	*COTWLTL* Pg. 59, Fig. 4-20 Pgs. 61-63, Sect. on *Exclusive Right-Turn Lanes* Pgs. 92-97, Intersct. Nos. 2 & 4 Pgs. 99-119, Intersct. Nos. 6-16 Pgs. 132-139, Intersct. Nos. 22-24 & 29 Pgs. 142-144, Intersct. Nos. 31-33 Pgs. 146-153, Intersct. Nos. 34-37	

As a driver approaches an intersection with the intention of traveling straight through, turning left, or turning right, he/she must first determine whether the currently traveled lane is the proper one for executing the intended maneuver. This understanding of the downstream intersection geometry is accomplished by the driver's visual search and successful detection, recognition, and comprehension of pavement markings (including stripes, symbols, and word markings); regulatory and/or advisory signs mounted overhead, in the median, and/or on the shoulder in advance of the intersection; and other geometric feature cues such as curb and pavement edge lines, pavement width transitions, and surface texture differences connoting shoulder or median areas. Uncertainty about downstream lane assignment produces hesitancy during the intersection approach; this in turn decreases available maneuver time and diminishes the driver's attentional resources available for effective response to potential traffic conflicts at and near intersections.

Older drivers' decreased contrast sensitivity, reduced useful field of view, increased decision time--particularly in response to unexpected events--and slower vehicle control during movement execution combine to put these highway users at greater crash risk when approaching and negotiating intersections. Contrast sensitivity and visual acuity are the visual/perceptual requirements necessary to detect pavement markings and symbols and to read lane control signs and word and symbol pavement markings. The early detection of lane control devices, by cueing the driver in advance that designated lanes exist for turning and through maneuvers, promotes safer and more confident performance of any required lane changes. This is because the traffic density is lighter, there are more available gaps, and there are fewer potential conflicts with other vehicles and pedestrians the farther away from the intersection the maneuver is performed. Of course, even the brightest delineation and pavement markings will not be visible to an operator unless an adequate sight distance (determined by horizontal and vertical alignment) is available.

In an effort to analyze the needs and concerns of older drivers, the Illinois Department of Transportation sponsored a statewide survey of 664 drivers, followed up by focus group meetings held in rural and urban areas (Benekohal, Resende, Shim, Michaels, and Weeks, 1992). Within this sample, the following four age categories were used for statistical analyses: ages 66-68, ages 69-72, ages 73-76, and age 77 and older. Comparisons of responses from drivers ages 66-68 and age 77 and older showed that the older group had more difficulty following pavement markings, finding the beginning of the left-turn lane, driving across intersections, and driving during daytime. Similarly, the level of difficulty for reading street signs and making left turns at intersections increased with increasing driver age. Turning left at intersections was perceived as a complex driving task, made more difficult when channelization providing visual cues was absent and only pavement markings designated which lane ahead was a through lane and which was a turning lane. The processes of lane location, detection, and selection must be made upstream at a distance where a lane change can be performed safely. Late detection by older drivers will result in erratic maneuvers such as lane weaving close to the intersection (McKnight and Stewart, 1990).

More than half of the 81 older drivers participating in more recent focus group discussions stated that quite often they suddenly find themselves in the wrong lane, because (1) they have certain expectations about lane use derived from intersections encountered earlier on the same roadway, (2) the advance signing is inadequate or lacking, or (3) the pavement markings are covered by cars at the intersection (Staplin, Harkey, Lococo, and Tarawneh, 1997). The biggest problem with turn-only lanes reported by group participants was that there is not enough warning for this feature. The appropriate amount of advance notice, as specified by these drivers, ranged from 5 car lengths to 1.6 km (1 mi). Sixty-four percent of the participants said that multiple warning signs are necessary when the right lane becomes a turn-only lane, with the need for an initial sign 20 to 30 s away, and a second sign 10 s away from the turn location. The remaining participants said that these distances should be increased.

Even greater consensus was shown in this study regarding sign location for lane assignment. Seventy-nine percent of the group reported that overhead lane-use signs are far more effective than roadside-mounted signs for this type of warning. Several participants suggested that a combination of roadside and overhead signs, in addition to roadway markings, would be beneficial. Although roadway markings were deemed helpful, 84 percent of all participants stated that they are useless in isolation from signs, because they are usually *at* the intersection and are obscured by traffic, and they are frequently worn and faded. The result is that drivers end up in the wrong lane and must go in a direction they had not planned for, or they try to change lanes at a point where it is not safe to do so. Thus, a general conclusion from this study is that overhead signing posted in advance of, as well as at, an intersection provides the most useful information to drivers about movement regulations which may be difficult to obtain from pavement marking arrows when traffic density is high or when pavement markings are obscured by snow or become faded, or where sight distance is limited.

In an early study conducted by Hoffman (1969), the installation of overhead lane-use control signs in advance of six intersections in Michigan contributed to a reduction in the total number of crashes by 44 percent in a 1-year period, and a reduction in the incidence of crashes caused by turning from the wrong lane by 58 percent. More recently, older drivers (as well as their younger counterparts) have been shown to benefit from redundant signing (Staplin and Fisk, 1991). In addition to redundant information about right-of-way movements at intersections, drivers should be forewarned about lane drops, shifts, and merges through advance warning signs, and ideally these conditions should not occur close to an intersection. Advance route or street signing as well as reassurance (confirmatory) signing/route marker assemblies across the intersection will aid drivers of all ages in deciding which lane will lead them to their destination, prior to reaching the intersection.

The MUTCD (2000) specifies in section 2B.18 that Intersection Lane Control signs should be mounted overhead, except where the number of through lanes for an approach is two or less, where the Intersection Lane Control signs (R3-5, R3-6, or R3-8) may be overhead or ground mounted. The Mandatory Movement signs (R3-5, R3-5a, and R3-7) are required to be located where the regulation applies. The Optional Movement Lane Control Sign (R3-6) is required to be located at the intersection. The MUTCD (2000) section on Advance Intersection Lane Control signs (sign series R3-8, section 2B.21), states that when used, these signs should be placed at an adequate distance in advance of the intersection so that road users can select the appropriate lane (e.g., in advance of the tapers or at the beginning of the turn lane). No guidance is provided regarding overhead vs ground mounting. Section 3B.19 indicates that where through lanes become mandatory turn lanes, signs or markings should be repeated as necessary to prevent entrapment and to help the road user select the most appropriate lane in advance of reaching a queue of waiting vehicles.

Although pavement markings have obvious limitations (e.g., limited durability when installed in areas exposed to heavy traffic, poor visibility on wet roads, and obscuration by snow in some regions), they have the advantage of presenting information to drivers without distracting their attention from the roadway.

Finally, the Institute of Transportation Engineers identified several features to enhance the operation of urban arterial trap lanes (through lanes that terminate in an unshadowed mandatory left- or right-turn regulation): (1) signing that gives prominent advance notice of the unexpected mandatory turn regulation, followed by a regulatory sign at the point where the mandatory turn regulation takes effect, followed by a third sign at the intersection itself if there are intervening driveways from which motorists might enter the lane; (2) supplemental pavement markings which consist of a double-width broken lane line beginning at the advance warning sign and extending to the first regulatory sign; (3) a pavement legend in the trap lane; and (4) overhead signing. Candidates for these remediations include left-turn trap lanes on roadways with high volumes, high speeds, poor approach visibility, and complex geometrics (Foxen, 1986).

N. Design Element: Traffic Signals

Table 21. Cross-references of related entries for traffic signals.

Applications in Standard Reference Manuals		
MUTCD (2000)	**AASHTO Green Book (1994)**	**Traffic Engineering Handbook (1999)**
Sect. 1A.13, *flashing (flashing mode)* & *traffic control signal (traffic signal)*	Pg. 76, Para. 4 Pg. 79, Para. 3 Pgs. 318-319, Sect. on *Signals* Pgs. 480-481, Sect. on *Traffic*	Pg. 455, 11th bullet Pg. 496, Sect. on *Visibility and Shielding*

Sects. 4A.02, 4D.01, 4D.04 through 4D.13, 4D.15 through 4D.18, 4E.03 through 4E.07, 4H.02, 4I.02, 4J.03, & 4K.01 MUTCD references to ITE standards ST-008B, ST-011B, ST-010 & Lane-Use Traffic Control Signal Heads	*Control Devices* Pgs. 534-535, Sect. on *Traffic Control Devices* Pgs. 716-718, Sect. on *Case IV-Signal Control* Pg. 637, Paras. 7 & 8 & Fig. IX-7 on pg. 640 Pg. 739, Paras. 4 & 5 Pg. 847, Para. 1 Pg. 939, Para. 3	

Traffic signals are power-operated signal displays used to regulate or warn traffic. They include displays for intersection control, flashing beacons, lane-directional signals, ramp-metering signals, pedestrian signals, railroad-crossing signals, and similar devices. Warrants for traffic signals are thoroughly described in the MUTCD. The decision to install a traffic signal is based on an investigation of physical and traffic flow conditions and data, including traffic volume, approach travel speeds, physical condition diagrams, crash history, and gap and delay information (Wilshire, 1992). The MUTCD incorporates the intensity, light distribution, and chromaticity standards from the following Institute of Transportation Engineers (ITE) standards for traffic signals: *Vehicle Control Signal Heads*, ITE Standard No ST-008B (ITE, 1985b); *Pedestrian Traffic Control Signal Indications*, ITE Standard No. ST-011B (ITE, 1985a); *Traffic Signal Lamps,* ITE Standard No. ST-010 (ITE, 1986); and *Lane-Use Traffic Control Signal Heads* (ITE, 1980). Standards for traffic signals are important because it is imperative that they attract the attention of every driver, including older drivers and those with impaired vision who meet legal requirements, as well as those who are fatigued or distracted, or who are not expecting to encounter a signal at a particular location. It is also necessary for traffic signals to meet motorists' needs under a wide range of conditions including bright sunlight, nighttime, in adverse weather, and in visually cluttered surroundings.

To date, studies of traffic signal performance have not typically included observer age as an independent variable. Available evidence suggests, however, that older individuals have reduced levels of sensitivity to intensity and contrast, but not to color. Fisher (1969) reported that as a person ages, the ocular media yellows and has the effect of enhancing the contrast between a red signal and a sky background. However, this effect is more than offset by increasing light scatter within the eye, which diminishes contrast. Older drivers need increased levels of signal luminance and contrast in certain situations to perceive traffic signals as efficiently as 20- to 25-year-old drivers; however, higher signal intensities may cause disability glare. Fisher and Cole (1974), using data from Blackwell (1970), suggested that older drivers may require 1.5 times the intensity at 50 years of age and 3 times the intensity at 70 years of age, and protanopes (individuals with a color-vision deficiency resulting in partial or full insensitivity to red light) may require a fourfold increase. They noted that while increased intensity will ensure that older observers see the signal, the reaction time of older drivers will be longer than for younger drivers. To compensate for this, it would appear necessary to assume a longer required visibility distance, which would result in an increase in the signal intensity required. However, Fisher (1969) also suggested that no increase in signal intensity is likely to compensate for increasing reaction time with age. It therefore deserves emphasis that the goal of increased response times for older drivers, requiring longer visibility distances, can also be provided by ensuring that the available signal strength (peak intensity) is maintained through a wide, versus a narrow, viewing angle. This makes signal information more accessible over longer intervals.

It is generally agreed that the visibility issues associated with circular signals relate to the following factors: minimum daytime intensity, intensity distribution, size, nighttime intensity, color of signals, backplates, depreciation (light loss due to lamp wear and dirt on lenses), and phantom (apparent illumination of a signal in a facing sun). To place this discussion in context, it should also be noted that traffic signal recommendations for different sizes, colors, and in-service requirements have, in large part, been derived analytically from one research study conducted by Cole and Brown (1966).

In establishing minimum daytime intensity levels for (circular) traffic signals, the two driver characteristics that are considered with regard to the need to adjust peak intensity requirements are color anomalies and driver age. Cole and Brown (1968) determined that the *optimum* red signal intensity is 200 cd for a sky luminance of 10,000 cd/m^2, and an *adequate* signal intensity for this condition would be 100 cd. Cole and Brown (1966, 1968) defined *optimum* as "a signal intensity that provides a very high probability of recognition and which also evokes the shortest response times from the observer." In their research, very high probability was defined as 95 to 100 percent probability of detection. An "adequate signal," although not likely to be missed, results in driver reaction time that is slower than for a signal of "optimum" intensity.

The number of foreign and domestic highway organizations that specify a minimum standard for peak daytime traffic signal intensity is larger than the number of research studies upon which those standards are based. In fact, all of the standards including those for 200-mm (8-in) and 300-mm (12-in) signals, those for red, yellow, and green signals, and those for new and in-service applications are derived from a single requirement for a red traffic signal, established from the work of Cole and Brown (1966). The conclusion of this laboratory study was that a red signal with an intensity of 200 cd should invoke a "certain and rapid response" from an observer viewing the signal at distances up to 100 m (328 ft) even under extremely bright ambient conditions. This conclusion was based on experiments in which the background luminance was 5,142 cd/m^2. The results were linearly extrapolated to a background luminance of 10,000 cd/m^2 which yielded the 200-cd recommendation. Janoff (1990) concluded that a value of 200 cd minimum intensity for a red signal will suffice for observation distances up to 100 m (328 ft) and vehicle speeds up to 80 km/h (50 mi/h), based on analytic, laboratory, and controlled field experiments performed by Adrian (1963); Boisson and Pages (1964); Rutley, Christie, and Fisher (1965); Jainski and Schmidt-Clausen (1967); Cole and Brown (1968); Fisher (1969); and Fisher and Cole (1974). Fisher and Cole (1974) cautioned against using a value less than 200 cd, to ensure that older drivers and drivers with abnormal color vision will see the signal with certainty and with "reasonable speed."

For green signals, Fisher and Cole (1974) indicated that the ratio of green to red intensity should be 1.33:1, based on laboratory and controlled field research by Adrian (1963), Rutley et al. (1965), Jainski and Schmidt-Clausen (1967), and Fisher (1969), and the ratio of yellow to red should be 3:1, based on research performed by Rutley et al. (1965) and Jainski and Schmidt-Clausen (1967). Janoff (1990) noted that the evidence to support these ratios is somewhat variable, and support of these recommendations is mixed. Table 22, from Janoff (1990), presents the peak intensity requirements of red, green, and yellow traffic signals for 200-mm (8-in) signals for normal-speed roads and for 300-mm (12-in) signals for high-speed roads; the values presented exclude the use of backplates and ignore depreciation. A normal-speed road, in this context, includes speeds up to 80 km/h (50 mi/h), distances up to 100 m (328 ft), and sky luminances up to 10,000 cd/m^2. A high-speed road is defined as one with speeds up to 100 km/h (62 mi/h), distances up to 240 m (787 ft), and sky luminances up to 10,000 cd/m^2. Janoff also noted that although signal size is included, research performed by Cole and Brown (1968) indicated that signal size is not important because traffic signals are point sources rather than area sources and only intensity affects visibility. Thus, the required intensity can be obtained by methods other than increasing signal size (i.e., by using higher intensity sources in 200-mm signals).

Table 22. Peak (minimum) daytime intensity requirement (cd) for maintained signals with no backplate. Source: Janoff, 1990.

Signal Size	Signal Color		
	Red	Green	Yellow
200 mm (8 in)	200	265	600
300 mm (12 in)	895	1,190	2,685

The specification of standard values for *peak* intensity is important because the distribution of light intensity falls off with increasing horizontal and vertical eccentricity in the viewing angle. Janoff (1990) summarized the peak intensity standards of ITE, Commission Internationale de l'Éclairage (CIE), the British Standards Organization, and standards organizations of Australia, Japan, and South Africa. The U.S. (ITE) standard provides different recommendations for each of the three colors for each signal size. The recommendations are as follows: for red, 157 cd for 200-mm (8-in) signals and 399 cd for 300-mm (12-in) signals; for green, 314 cd for 200-mm (8-in) signals and 798 cd for 300-mm (12-in) signals; and for yellow, 726 cd for 200-mm (8-in) signals and 1,848 cd for 300-mm (12-in) signals. Australia recommends the same peak intensity for red and green (200 cd for 200-mm [8-in] signals and 600 cd for 300-mm [12-in] signals), and a yellow intensity equal to three times the red intensity. The CIE recommends the same peak intensity for all three colors (200 cd for 200-mm [8-in] signals and 600 cd for 300-mm [12-in] signals), but acknowledges that actual intensity differences between colors result due to the differential transmittance of the colored lenses (1:1.3 for red to green and 1:3 for red to yellow). Japan recommends 240 cd for all three colors. Great Britain recommends a peak intensity of 475 cd for 200-mm (8-in) red and green signals, and 800 cd for 300-mm (12-in) red and green signals. The range for red signals among all of these standards is from 157 cd (ITE) to 475 cd (British Standards Organization). The 157 cd is from research by Cole and Brown. The modal value of 200 cd, specified by Australia, South Africa, and the CIE, is based upon a depreciation factor of 33 percent.

Only two research reports provide intensity requirements for green and/or yellow signals based upon empirical data. Adrian (1963) used a subjective scale and threshold detection criteria in a study that tested red and green signals at different background luminances. He concluded that the intensity requirements for green were 1.0 and 1.2 times that of red for the subjective and threshold studies, respectively. Jainski and Schmidt-Clausen (1967) tested the ability of observers to detect the presence of a red, amber, or green spot, which was either 2 minutes or 1 degree, against varying background luminances. Their results found that green required 1.0 and 2.5 times that of red, and yellow required 2.5 and 3.0 times that of red, for 1 degree and 2 minutes, respectively. Using these results, most standards set requirements for green and yellow to be 1.3 and 3.0 times that of red, respectively. The CIE standard discusses the fact that the ratios of 1.3 and 3.0 for green and yellow appear to reflect the differences in the transmissivity of the varying color lenses.

The most current information on signal intensity requirements that will accommodate road users with age-related vision deficiencies is provided by NCHRP Project 5-15, *Visibility Performance Requirements for Vehicular Traffic Signals*. This investigation includes a series of laboratory and field studies to determine performance-based signal requirements for traffic signal intensity, intensity distribution, and related photometric parameters using a subject population that oversamples older drivers (Freedman, Flicker, Janoff, Schwab, and Staplin, 1997). While the final results and recommendations from this research were not yet published when this Handbook was prepared, one preliminary finding deserves emphasis: minimum daytime brightness requirements must be stated in terms of *maintained* signal performance levels. The present recommendation in this area accordingly augments the 200 cd intensity requirement for red 200-mm (8-in) signals that appears most prominently in the literature cited above (e.g., Janoff, 1990) with this emphasis on in-service performance measurement.

Holowachuk, Leung, and Lakowski (1993) conducted a laboratory study to evaluate the effects of color vision deficiencies and age-related diminished visual capability on the visibility of traffic signals. Subjects ranged in age from 18 to 80 and older, and included 64 individuals with normal color vision and 51 subjects who were color-vision deficient. A laboratory simulation apparatus was used to present photographs taken of seven signal head assemblies at intersections at distances of 50 and 100 m (164 and 328 ft). The photographs were taken at intersections in the Vancouver area within simple and complex environments. Each subject viewed 48 photographs shot during daylight conditions and 38 photographs shot at nighttime. Subjects' reaction times to recognize the color of the "on" signal were measured, as was the accuracy of response. The basic highway signal head used by the Ministry of Transportation and Highways in British Columbia consists of a 300-mm (12-in) red light, a 200-mm (8-in) amber light, and a 200-mm (8-in) green light arranged vertically with a yellow backplate. This "standard highway" signal plus six other off-the-shelf signal-head designs were used in the study, as shown in table 23.

Results indicated that color-vision deficient drivers had significantly longer reaction times than drivers with normal color vision, and older drivers had longer reaction times compared to younger drivers. Of particular importance is that the reaction times of the normal color vision drivers over age 50 (n=15) compared closely to those of color vision deficient drivers (n=50). Regarding signal design, for daytime conditions, the no backplate assembly produced the longest reaction times for both the normal color vision and the color-vision deficient drivers. Reaction times for the larger and brighter lenses (shape coded and 300 RYG) were the shortest, for both groups of subjects. For nighttime conditions, the signal assemblies showed few differences in reaction time for subjects with normal color vision. Reaction times were shortest for the shape coded and 300 RYG assemblies, however the baseline assembly and the No Backplate assemblies produced the longest reaction times. For the color-vision deficient group, the reaction times for the shape coded, 300 RYG, and the Modified Backplate assemblies were distinctly shorter than those for the Baseline and No Backplate assemblies. Nighttime reaction times were much longer than daytime reaction times for the subjects with color vision deficiencies. Signal light colors were identified more incorrectly for night conditions than for day conditions. This difference was greatest for the older color vision deficient drivers (n=22).

Table 23. Signal Head Designs Evaluated by Holowachuk et al. (1993, 1994).

Name	Abbreviation	Lens Size (mm)*	Backplate	Other Features
No Backplate	NO BP	Red 200, Amber 200, Green 200	No	N/A
Base Line	Baseline	Red 200, Amber 200, Green 200	Yes	N/A
Modified Backplate	Mod BP	Red 200, Amber 200, Green 200	Yes	Backplate with 50 mm reflective border
Standard Highway	Std Hwy	Red 300, Amber 200, Green 200	Yes	N/A
300 mm LED	LED	Red 300 (LED), Amber 200, Green 200	Yes	300 mm red LED signal
300 mm RED, Green, Amber	300 RYG	Red 300, Amber 300, Green 300	Yes	N/A
300 mm Shape	Shape Coded	Red 300, Amber 300,	Yes	Red Square

Coded		Green 300		Amber Diamond Red Circle

***Note:** 300-mm lens uses 150-Watt bulb; 200-mm lens uses 69-Watt bulb.

** N/A-Not applicable

Overall, findings indicated that the reaction times for all subjects were the shortest for signal designs with larger lenses (300 mm) and higher luminances (150-W bulbs). There was no significant difference in reaction times between the shape coded and the 300 RYG, for the normal subjects or for the color-vision deficient subjects. The next-best performing signal design was the Modified Backplate. The signal assembly with no backplate produced the longest reaction times. Based on these findings, a new signal specification was established for field testing, consisting of all 300-mm signal lenses and a backplate with an additional 75 mm of reflective border. This new assembly has been under test since 1994 at 10 treatment and 10 control intersections located on major highway corridors in Burnaby, Maple Ridge, Surrey, and Saanich (British Columbia, Canada). In a preliminary evaluation, the total number of collisions was reduced by 24 percent as a result of the new signal head design, and the severity was reduced by 20 percent. (Leung, in progress).

Some research has indicated that the dimming of signals at night may have advantages, while also reducing power consumption. Freedman, Davit, Staplin, and Breton (1985) conducted a laboratory study and controlled and observational field studies to determine the operational, safety, and economic impact of dimming traffic signals at night. Results indicated that drivers behaved safely and efficiently when signals were dimmed to as low as 30 percent of ITE recommendations. Previously, however, Lunenfeld (1977) cited the considerable range of night background luminances that may occur in concluding that in some brightly lit urban conditions, or where there is considerable visual noise, daytime signal brightness is needed to maintain an acceptable contrast ratio. The ITE standard presently does not differentiate between day and night intensity requirements. The CIE recommends that intensities greater than 200 cd or less than 25 cd be avoided at night and advises a range of 50 to 100 cd for night, except for high-speed roads where the daytime values are preferred. The South African and Australian standards allow for dimming but do not recommend an intensity level. While the option for dimming on a location-by-location basis should not be excluded, from the standpoint of older driver needs, there is no compelling reason to recommend widespread reduction of traffic signal intensity during nighttime operations.

It is common practice to try to enhance the visibility of signals by placing a large, black surround behind the signals. The backplate, rather than the sky, becomes the background of the signals, enhancing the contrast. Regarding backplate size, no recommendation is contained in the ITE standard. The CIE (1988), however, recommends that all signals use backplates of a size (width) of three times the diameter of the signal. As a practical matter, the use of a backplate also serves to compensate, in part, for the effects of depreciation, since a backplate reduces the required intensity by roughly 25 percent (Cole and Brown, 1966) while depreciation increases the requirement by the same amount. Guidelines published by the CIE (1988) include an allowance of 25-percent transmissivity for depreciation due to dirt and aging (a 33-percent increase in intensity for new installations). The 200-cd requirement for red signals, as noted earlier, must be met *after* the depreciation factor has been taken into account.

Regarding signal size, section 4D.15 of the MUTCD specifies that the two nominal diameter sizes for vehicular signal lenses are 200-mm (8-in) and 300-mm (12-in), and provides guidance that states that 300-mm (12-in) lenses should be used at locations where there is a significant percentage of older drivers. Researchers at the Texas Transportation Institute propose that the larger 300-mm (12-in) lens should be used to improve the attention-getting value of signals for older drivers (Greene, Koppa, Rodriguez, and Wright, 1996). Use of the large lens also provides motorists with more time to determine the signal color and to make the correct response.

A final issue with respect to signal performance and older drivers is the change intervals between phases, and the assumptions about perception-reaction time (PRT) on which these calculations are based. At present, a value of 1.0 s is assumed to compute change intervals for traffic signals, a value which, according to Tarawneh (1991), dates back to a 1934 Massachusetts Institute of Technology study on brake-reaction time. Tarawneh examined findings published by proponents of both "parallel" and "sequential" (serial) models of driver information processing, seeking to determine the best estimator for older individuals of a PRT encompassing six different component processing operations: (1) latency time (onset of stimulus to beginning of eye movement toward signal); (2) eye/head movement time to fixate on the signal; (3) fixation time to get enough information to identify the stimulus; (4) recognition time (interpret signal display in terms of possible courses of action); (5) decision time to select the best response in the situation; and (6) limb movement time to accomplish the appropriate steering and brake/accelerator movements.

Tarawneh's (1991) review produced several conclusions. First, the situation of a signal change at an intersection is among the most extreme, in terms of both the information-processing demand and subjective feelings of stress that will be experienced by many older drivers. Second, the most reasonable interpretation of research to date indicates that the best "mental model" to describe and predict how drivers respond in this context includes a mix of concurrent and serial-and-contingent information-processing operations. In this approach, the most valid PRT estimator will fall between the bounds of values derived from the competing models thus far, also taking into account age-related response slowing for recognition, decision-making, and limb movement. After a tabular summary of the specific component values upon which he based his calculations, Tarawneh (1991) called for an increase in the current PRT value used to calculate the length of the yellow interval (derived from tests of much younger subjects) from 1.0 s to 1.5 s to accommodate older drivers.

A contrasting set of results was obtained in a recent FHWA-sponsored study of traffic operations control for older drivers (Knoblauch, Nitzburg, Reinfurt, Council, Zegeer, and Popkin, 1995). This study compared the decision/response times and deceleration characteristics of older drivers (ages 60-71 and older) with those of younger drivers (younger than age 60) at the onset of the amber signal phase. Testing was conducted using a controlled field test facility, where subjects drove their own vehicles. Subjects were asked to maintain speeds of 48 km/h (30 mi/h) and 32 km/h (20 mi/h) for certain test circuits. The duration of the yellow signal was 3.0 s before turning to red. On half of the trials, the signal changed from green to yellow when the subject was 3.0 to 3.9 s from the signal, and on the remaining trials, when the subject was 4.0 to 4.9 s away from the signal. For three of the circuits, subjects were asked to brake as they normally would and to stop before reaching the intersection, if they chose to do so. During a fourth circuit, they were asked to brake to a stop, if they possibly could, if the light changed from green to yellow. Response times were measured for the drivers who stopped, from the onset of the yellow phase to the time the brake was applied.

Results of the Knoblauch et al. (1995) study showed no significant differences in 85th percentile decision/response times between younger and older drivers when subjects were close to the signal at either approach speed. The 85th percentile decision time of younger subjects was 0.39 s at 32 km/h (20 mi/h) and 0.45 s at 48 km/h (30 mi/h). For older drivers, these times were 0.51 and 0.53 s, for 32 km/h and 48 km/h (20 mi/h and 30 mi/h), respectively. When subjects were further from the signal at amber onset, older drivers had significantly longer decision/response times (1.38 s at 32 km/h [20 mi/h] and 0.88 s at 48 km/h [30 mi/h]) than the younger drivers (0.50 s at 32 km/h [20 mi/h] and 0.46 s at 48 km/h [30 mi/h]). The authors suggested that the significant differences between older and younger drivers occurred when the subjects were relatively far from the signal, and that some older subjects will take longer to react and respond when additional time is available for them to do so. Thus, they concluded that the older drivers were not necessarily reacting inappropriately to the signal. In terms of deceleration rates, there were no significant differences, either in the mean or 15th percentile values, between the older and younger subjects. Together, these findings led the authors to conclude that no changes in amber signal phase timing are required to accommodate older drivers.

Taking the review and study findings of Tarawneh (1991) and Knoblauch et al. (1995) into consideration, an approach that retains the 1.0-s PRT value as a minimum for calculating the yellow change interval seems appropriate; but, to acknowledge the significant body of work documenting age-related increases in PRT, the use of a 1.5-s PRT is well justified when engineering judgment determines a special need to take older drivers' diminished capabilities into account. A recommendation for an all-red clearance interval logically follows, with length determined according to the ITE (1992).

O. Design Element: Fixed Lighting Installations

Table 24. Cross-references of related entries for fixed lighting installations.

Applications in Standard Reference Manuals		
MUTCD (2000)	**AASHTO Green Book (1994)**	**Roadway Lighting Handbook (1978)**
Sect. 1A.13, *sign illumination* Sects. 2A.08, 2E.05, & 3G.04 Sect. 4B.04 Sects. 6D.01 & 6D.02 Sect. 6F.70 Sect. 6G.13	Pgs. 309-311, Sect. on *Lighting* Pg. 315, Para. 2 Pgs. 440-441, Sect. on *Street and Roadway Lighting* Pg. 480, Sect. on *Street and Roadway Lighting*	Pgs. 16-27, Sects. on *Analytical Approach to Illumination Warrants, Informational Needs Approach to Warrants,* & *Warrants for Rural Intersection Lighting* Pgs. 29-30 Sect. on *Adverse Geometry and Environment Warrant* Pgs. 42-45, Sect. on *Summary of Light Sources* Pgs. 53-56, Sect. on *Classification of Luminaire Light*

Table 6H-2 Figs. 6H-12, 6H-40, 6H-41 Sects. 8C.01, 10C.14	Pgs. 545-546, Sect. on *Lighting* Pg. 567, Para. 1 & Fig. VIII-6 on pg. 570 Pg. 792, Sect. on *Lighting at Intersections*	*Distributions* Pg. 71, 5th bullet Pg. 94, Sect. on *Coordination of the Arterial Lighting System and Traffic Controls* Pg. 96, Sect. on *Intersection Lighting* Pgs. 98-99, Sect. on *Rural Intersection Lighting* Pgs. 120-129, Sect. on *Illumination Design Procedure* Pgs.187-200, Sect. on *Maintaining the System*

One of the main purposes of lighting a roadway at night is to increase the visibility of the roadway and its immediate environment, thereby permitting the driver to maneuver more safely and efficiently. The visibility of an object is that property which makes it discernible from its surroundings. This property depends on a combination of factors; principally, these factors include the differences in luminance, hue, and saturation between the object and its immediate background (contrast); the angular size of the object at the eye of the observer; the luminance of the background against which it is seen; and the duration of observation.

Of all the highway safety improvement projects evaluated by FHWA (1996), using data from 1974 to 1995 where before- and after-exposure data were available, intersection illumination was associated with the highest benefit-cost ratio (26.8) in reducing fatal and injury crashes. The link between reduced visibility and highway safety is conceptually straightforward. Low luminance contributes to a reduction in visual capabilities such as acuity, distance judgment, speed of seeing, color discrimination, and glare tolerance, which are already diminished capabilities in older drivers.

The Commission Internationale de l'Éclairage (1990) reports that road crashes at night are disproportionately higher in number and severity compared with crashes during the daytime. Data from 13 Organization for Economic Cooperation and Development countries showed that the proportion of fatal nighttime crashes ranged between 25 and 59 percent (average value of 48.5 percent). In this evaluation of 62 lighting and crash studies, 85 percent of the results showed lighting to be beneficial, with approximately one-third of the results statistically significant.

In 1990, drivers (without regard to age) in the United States experienced 10.37 fatal involvements per 161 million km (100 million mi) at night and 2.25 fatal involvements per 61 million km (100 million mi) during the day (Massie and Campbell, 1993). In their analysis, the difference between daytime and nighttime fatal rates was found to be more pronounced among younger age groups than among older ones, with drivers ages 20-24 showing a nighttime rate that was 6.1 times the daytime rate, and drivers age 75 and older showing a nighttime rate only 1.1 times the daytime rate. The lower percentage of nighttime crashes of older drivers may be due to a number of factors, including reduced exposure--older drivers as a group drive less at night--and a self-regulation process whereby those who do drive at night are the most fit and capable to perform all functional requirements of the driving task (National Highway Traffic Safety Administration, 1987).

A specific driving error with high potential for crash involvement is wrong-way movements. Analyses of wrong-way movements at intersections frequently associate these driving errors with low visibility and restricted sight distance (Vaswani, 1974, 1977; Scifres and Loutzenheiser, 1975) and note that designs that increase the visibility of access points to divided highways and treatments that improve drivers' understanding of proper movements at these locations have been found to reduce the potential for crashes.

Inadequate night visibility, where the vehicle's headlights are the driver's primary light source, was reported by Vaswani (1977) as a factor that makes it more difficult for drivers to determine the correct routing at intersections with divided highways. Similarly, Woods, Rowan, and Johnson (1970) reported that locations where highway structures, land use, natural growth, or poor lighting conditions reduce the principal sources of information concerning the geometry and pavement markings are associated with higher occurrences of wrong-way maneuvers. Crowley and Seguin (1986) reported that drivers over the age of 60 are excessively involved in wrong-way movements on a per-mile basis. Suggested countermeasures include increased use of fixed lighting installations. Lighting provides a particular benefit to older drivers by increasing expectancy of needed vehicle control actions, at longer preview distances. It has been documented extensively in this Handbook that an older driver's ability to safely execute a *planned* action is not significantly worse than that of a younger driver. The importance of fixed lighting at intersections for older drivers can therefore be understood in terms of both the diminished visual capabilities of this group and their special needs to prepare farther in advance for unusual or unexpected aspects of intersection operations or geometry. Targets that are especially critical in this regard include shifting lane alignments; changing lane assignments (e.g., when a through lane changes to turn-only operation); a pavement width transition, particularly a reduction across the intersection; and, of course, pedestrians.

Detectability of a pedestrian is generally influenced by contrast, motion, color, and size (Robertson, Berger, and Pain, 1977). If a pedestrian is walking at night and does not have good contrast, color contrast, or size relative to other road objects, an increase in contrast will significantly improve his/her detectability. This is one effect of street lighting. Extreme contrasts as well as dark spots are reduced, giving the driver and the pedestrian a more "even" visual field. The effectiveness of fixed lighting in improving the detectability of pedestrians has been reported by Pegrum (1972); Freedman, Janoff, Koth, and McCunney (1975); Polus and Katz (1978); and Zegeer (1991).

While age-related changes in glare susceptibility and contrast threshold are currently accounted for in lighting design criteria, there are other visual effects of aging that are currently excluded from visibility criteria. Solid documentation exists of age-related declines in ocular transmittance (the total amount of light reaching the retina), particularly for the shorter wavelengths (cf. Ruddock, 1965); this suggests a potential benefit to older drivers of the "yellow tint" of high-pressure sodium highway lighting installations. The older eye experiences exaggerated intraocular scatter of light--all light, independent of wavelength (Wooten and Geri, 1987)--making these drivers more susceptible to glare. Diminished capability for visual accommodation makes it harder for older observers to focus on objects at different distances. Pupil size is reduced among older individuals through senile miosis (Owsley, 1987), which is most detrimental at night because the reduction in light entering the eye compounds the problem of light lost via the ocular media, as described above.

The loss of static and dynamic acuity--the ability to detect fine detail in stationary and moving targets--with advancing age is widely understood. Although there are pronounced individual differences in the amount of age-related reduction in static visual acuity, Owsley (1987) indicated that a loss of about 70 percent in this capability by age 85 is normal. Among other things, declines in acuity can be used to predict the distance at which text of varying size can be read on highway signs (Kline and Fuchs, 1993), under a given set of viewing conditions.

There are a number of other aspects of vision and visual attention that relate to driving. In particular, saccadic fixation, useful field of view, detection of motion in depth, and detection of angular movement have been shown to be correlated with driving performance (see Bailey and Sheedy, 1988, for a review). As a group, however, these visual functions do not appear to have strong implications for highway lighting practice, with the possible exception of the "useful field of view." It could be argued that it would be advantageous to provide wider angle lighting coverage to increase the total field of view of older drivers. High-mast lighting systems can increase the field of view from 30 degrees to about 105 degrees (Hans, 1993). Such wide angles of coverage might have advantages for older drivers in terms of peripheral object detection. However, because high-mast lighting systems tend to sacrifice target contrast for increased field of view, opinion is divided about their application at intersections. Currently, field of view is not considered as a parameter that needs to be optimized in lighting system design for intersection applications.

Rockwell, Hungerford, and Balasubramanian (1976) studied the performance of drivers approaching four intersection treatments, differentiated in terms of special reflectorized delineators and signs versus illumination. A significant finding from observing 168 test approaches was that the use of roadway lighting significantly improved driving performance and earlier detection of the intersection, compared with the other treatments (e.g., signing, delineation, and new pavement markings), which showed smaller improvements in performance.

Finally, it must be emphasized that the effectiveness of intersection lighting depends upon a continuing program of monitoring and maintenance by the local authority. Guidelines published by AASHTO (1984) identify depreciation due to dirt on the luminaires and reduced lumen output from the in-service aging of lamps as factors that combine to decrease lighting system performance below design values. Maintained values in the range of 60 to 80 percent of initial design values are cited as common practice in this publication. With a particular focus on the needs of older drivers for increased illumination relative to younger motorists, to accommodate the age-related sensory deficits documented earlier in this discussion, a recommendation logically follows that lighting systems be maintained to provide service at the 80 percent level--i.e., the upper end of the practical range--with respect to their initial design values.

P. Design Element: Pedestrian Crossing Design, Operations, and Control

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Table 25. Cross-references of related entries for pedestrian crossing design, operations, and control.

Applications in Standard Reference Manuals				
MUTCD (2000)	**AASHTO**	**Roadway Lighting**	**NCHRP 279**	**Traffic Eng.**

	Green Book (1994)	**Handbook (1978)**	**Intersection Channelization Design Guide (1985)**	**Hndbk. (1999)**
Sect. 1A.13,*crosswalk, crosswalk lines, & pedestrian* Tables 2A-1 & 2C-2 through 2C-3 Sects. 1A.14, 2B.39, 2B.40, 2C.37, 3B.15 & 3B.17 Figs. 3B-15 & 3B-16 Sect. 3B.19 Sect. 3E.01 Sect. 4A.02 Sects. 4C.05 & 4D.03 Sect. 4D.16 Sects. 4E.01 through 4E.09 Fig. 4E-1 Sect. 7A.03 Fig. 7A-1 Table 7B-1 Sects. 7B.08 & 7B.09 Fig. 7B-1 Sects. 7C.03, 7E.01 through 7E.09, 10C.01 & 10D.08	Pgs. 97-104, Sects. on*General Considerations*, *General Characteristics*, *Physical Characteristics*, *Walk-way Capacities*, *Intersections*, *Characteristics of Persons with Disabilities*, & *Conclusions* Pg. 350, Para. 5 Pgs. 390-403, Sects. on *Separations for Pedestrian Crossings* & *Curb-Cut Ramps* Pg. 437, Sect. 0n *Curb-Cut Ramps* Pgs. 531-532 Sect. on *Pedestrian Facilities* & *Curb-Cut Ramps* Pg. 664, Para. 4 Pgs. 668-672, Sect. on *Effect of Curb Radii on Turning Paths* Pg. 679, Sect. on *Refuge Islands* Pg. 792, Sect. on *Wheel-Chair Ramps at Intersections*	Pg. 2, 2nd col., Para. 1 Pg. 9, Sect. on *Contrast* Pg. 18, Form 2 Pgs. 21-22, Tables 1-2 Pgs. 27-30, Sect. on *Warrants for Application of Specialized Crosswalk Illumination* Pg. 45, Para. 3 Pg. 71, 3rd bullet Pgs. 94-9, Sects. on *Coordination of the Arterial Lighting System and Traffic Controls* & *Pedestrian Lighting on Arterials*	Pg. 1, Item 3, 2nd bullet Pg. 6, Table 2-2 Pg. 9, Fig. 2-4 Pg. 11, Bottom left photo Pg. 21, Item 9 & Fig. 3-1 Pg. 27, Bottom left fig. Pg. 33, Bottom right fig. Pg. 38, Para. 1 & top two figs. Pg. 39, Entire pg. Pg. 40, Fig. 4-1 Pg. 42, 2nd col., item 3 Pg. 61, 2nd col, Paras. 2-3 & item 2 Pg. 66, Paras. 1 & 4 & item 4 Pg. 67, 2nd col, Paras. 1-2 Pg. 68, Fig. 4-26 Pg. 69, 2nd col., Para. 4 Pgs. 70-72, Figs. 4-27 & 4-28 Pg. 75, Paras. 4-5, Table 4-10, & 2nd col, Para. 1 Pg. 76, Item 4 & Table 4-11 Pgs. 96-97, Intersct. No. 4 Pgs. 104-105, Intersct. No. 9 Pgs. 150-151, Intersct. No. 36	Pgs. 41-42, Sect. on *Walking Speed* Pg. 43, Sect. on *Handicapped Pedestrians* Pgs. 46-47, Sect. on *Older Pedestrians* Pg. 385, Item 9 Pg. 409, Sect. on *Intersection Treatments* Pg. 414, Item 5 Pgs. 434-435, Sect. on *Crosswalks* Pgs. 497-498, Sect. on *Pedestrian Signal Heads* Pgs. 519-520, Sect. on *Pedestrian Detectors*

A nationwide review of fatalities during the year 1985, and injuries during the period of 1983-1985, showed that 39 percent of all pedestrian fatalities and 9 percent of all pedestrian injuries involved persons age 64 and older (Hauer, 1988). While the number of injuries is close to the population distribution (approximately 12 percent), the number of fatalities far exceeds the proportion of older pedestrians. The percentages of pedestrian fatalities and injuries occurring at intersections were 33 percent and 51 percent, respectively (Hauer, 1988). People age 80 and older have the highest pedestrian death rate per 100,000 people; furthermore, the 1998 pedestrian death rate among men age 80 and older was more than 3 times as high as that for men age 74 and younger (IIHS, 2000). Crash types that predominantly involve older pedestrians at intersections are as follows (Blomberg and Edwards, 1990):

- Vehicle turn/merge--The vehicle turns left or right and strikes the pedestrian.
- Intersection dash--A pedestrian appears suddenly in the street in front of an oncoming vehicle at an intersection.
- Multiple threat--One or more vehicles stop in the through lane, usually at a crosswalk at an unsignalized intersection. The pedestrian steps in front of the stopped vehicle(s) and into the path of a through vehicle in the adjacent lane.
- Bus-stop related--The pedestrian steps out from in front of a stopped bus and is struck by a vehicle moving in the same direction as the bus.

- Pedestrian trapped--At a signalized intersection, a pedestrian is hit when a traffic signal turns red (for the pedestrian) and cross-traffic vehicles start moving.
- Nighttime--A pedestrian is struck at night when crossing at an intersection.

Earlier analyses of over 5,300 pedestrian crashes occurring at urban intersections indicated that a significantly greater proportion of pedestrians age 65 and older were hit at signalized intersections than any other group (Robertson, Berger, and Pain, 1977).

Age-related diminished capabilities, which may make it more difficult for older pedestrians to negotiate intersections, include decreased contrast sensitivity and visual acuity, reduced peripheral vision and "useful field of view," decreased ability to judge safe gaps, slowed walking speed, and physical limitations resulting from arthritis and other health problems. Older pedestrian problem behaviors include a greater likelihood to delay before crossing, to spend more time at the curb, to take longer to cross the road, and to make more head movements before and during crossing (Wilson and Grayson, 1980).

Older and Grayson (1972) reported that although older pedestrians involved in crashes looked more often than the middle-aged group studied, over 70 percent of the adults struck by a vehicle reported not seeing it before impact. Job, Haynes, Prabhaker, Lee, and Quach (1992) found that pedestrians over age 65 looked less often *during* their crossings than did younger pedestrians. In a survey of older pedestrians (average age of 75) involved in crashes, 63 percent reported that they failed to see the vehicle that hit them, or to see it in time to take evasive action (Sheppard and Pattinson, 1986). Knoblauch, Nitzburg, Dewar, Templer, and Pietrucha (1995) noted that difficulty seeing a vehicle against a (complex) street background may occur with vehicles of certain colors, causing them to blend in with their background. This is especially problematic for older persons with reduced contrast sensitivity, who require a higher contrast for detection of the same targets than younger individuals, and who also have greater difficulty dividing attention between multiple sources and selectively attending to the most relevant targets. In addition, the loss of peripheral vision increases an older pedestrian's chances of not detecting approaching and turning vehicles from the side.

Reductions in visual acuity make it more difficult for older pedestrians to read the crossing signal. In a survey of older pedestrians in the Orlando, Florida area, 25 percent of the participants reported difficulty seeing the crosswalk signal from the opposite side of the street (Bailey, Jones, Stout, Bailey, Kass, and Morgan, 1992). Older pedestrians wait for longer gaps between vehicles before attempting to cross the road. In one study, approximately 85 percent of the pedestrians age 60 and older required a minimum gap of 9 s before crossing the road, while only 63 percent of all pedestrians required this minimum gap size duration (Tobey, Shungman, and Knoblauch, 1983). The decline in depth perception may contribute to older persons' reduced ability to judge gaps in oncoming traffic. It may be concluded from these studies that older pedestrians do not process information (presence, speed, and distance of other vehicles) as efficiently as younger pedestrians, and therefore require more time to reach a decision. Other researchers have observed that older pedestrians do not plan their traffic behavior, are too trusting about traffic rules, fail to check for oncoming traffic before crossing at intersections, underestimate the speed of approaching vehicles, and follow other pedestrians without first checking for conflicts before crossing (Jonah and Engel, 1983; Mathey, 1983).

With increasing age, there is a concurrent loss of physical strength, joint flexibility, agility, balance, coordination and motor skills, and stamina. These losses contribute to slower walking speeds and difficulty negotiating curbs. In addition, older persons often fall as a result of undetected surface irregularities in the pavement and misestimation of curb heights. This results from a decline in contrast sensitivity and depth perception. In an assessment of 81 older residents (ages 70-97) to examine susceptibility to falling, 58 percent experienced a fall in the year following clinical assessment (Clark, Lord, and Webster, 1993). Impaired cognition, abnormal reaction to any push or pressure, history of palpitations, and abnormal stepping were each associated with falling. Knoblauch, Nitzburg, Reinfurt, Council, Zegeer, and Popkin (1995) reported that locating the curb accurately and placing the foot is a matter of some care, particularly for the elderly, the very young, and those with physical disabilities.

The studies discussed below define the types of crashes in which older pedestrians are most likely to be involved, and under what conditions the crashes most frequently occur. In addition, the specific geometric characteristics, traffic control devices (including signs, signals, and markings), and pedestrian signals that seem to contribute to older pedestrians' difficulties at intersections are discussed. Zegeer and Zegeer (1988) stressed the importance of "tailoring" the most appropriate traffic control measures to suit the conditions at a given site. The effect of any traffic control measure is highly dependent on specific locational characteristics, such as traffic conditions (e.g., volumes, speeds, turning movements), pedestrian volumes and pedestrian mix (e.g., young children, college students, older adults, persons with physical disabilities), street width, existing traffic controls, area type (e.g., rural, urban, suburban), site distance, crash patterns, presence of enforcement, and numerous other factors.

Harrell (1990) used distance stood from the curb as a measure of pedestrian risk for intersection crossing. Observations of 696 pedestrians divided among 3 age groups (age 30 and under, ages 31-50, and age 51 and older) showed that the oldest group stood the farthest from the curb, that they stood even farther back under nighttime conditions, and that older females stood the farthest distance from the curb. The author used these data to dispel the findings in the literature that older pedestrians are not cognizant of the risks of exposure to injury from passing vehicles. Similarly, it may be argued that this behavior keeps them from detecting potential conflict vehicles and makes speed and distance judgments more difficult for them, while limiting their conspicuity to approaching drivers who might otherwise slow down if pedestrians were detected standing at the curbside at a crosswalk.

A study of pedestrian crashes conducted at 31 high-pedestrian crash sections in Maryland between 1974 and 1976 showed that pedestrians age 60 and older were involved in 53 (9.6 percent) of the crashes, and children younger than age 12 showed the same proportions. The pedestrians age 60 and older accounted for 25.6 percent of the fatal crashes. Compliance with traffic control devices was found to be poor for all pedestrians at all study locations; it was also found that most pedestrians keyed on the moving vehicle rather than on the traffic and pedestrian control devices. Only when the traffic volumes were so high that it was impossible to cross did pedestrians rely on traffic control devices (Bush, 1986).

Garber and Srinivasan (1991) conducted a study of 2,550 crashes involving pedestrians that occurred in the rural and urban areas of Virginia to identify intersection geometric characteristics and intersection traffic control devices that were predominant in crashes involving older pedestrians. Crash frequency by location and age for the crashes within the cities showed that while the highest percentage of crashes involving pedestrians age 59 and younger occurred within 46 m (150 ft) from the intersection stop line, the highest percentage of crashes for pedestrians age 60 years and older (51.8 percent) occurred *within the intersection*.

More recently, Knoblauch, Nitzburg, Reinfurt, et al. (1995) reported that, compared with younger pedestrians, older adults are overinvolved in crashes while crossing streets at intersections. In their earlier analysis of the national Fatal Analysis Reporting System (FARS) data for the period 1980-1989, 32.2 and 35.3 percent of the deaths for pedestrians ages 65-74 and age 75 and older, respectively, occurred at intersections (Reinfurt, Council, Zegeer, and Popkin, 1992). This compared with 22 percent or less for the younger age groups. Analysis of the North Carolina motor vehicle crash file for 1980-1990 displayed somewhat smaller percentages, but showed the trend of increasing pedestrian crashes at intersections as age increased. Further analysis of the North Carolina database showed that pedestrians age 65 and older as well as those ages 45-64 experienced 37 percent of their crashes on roadways with four or more lanes. This compares with 23.7 percent for pedestrians ages 10-44 and 13.6 percent for those age 9 and younger. The highest number of pedestrian-vehicle crashes occurred when the vehicle was going straight (59.7 percent), followed by a vehicle turning left (17.2 percent), and a vehicle turning right (13.3 percent). Right-turn crashes accounted for 18.9 percent of crashes with pedestrians ages 65-74, compared with 14.2 percent for pedestrians age 75 and older. The oldest pedestrian group was the most likely to be struck by a left-turning vehicle; they accounted for 23.9 percent of the crashes, compared with 18.1 percent of those ages 65-74 and 15.8 percent of those ages 45-64.

Knoblauch, Nitzburg, Reinfurt, et al. (1995) conducted a study to determine if pedestrian comprehension of and compliance with pedestrian signals could be improved by installing a placard that explained the three phases of pedestrian signals. They used findings from: (1) a focus group and workshop conducted in Baltimore, Maryland, with 13 participants ages 19-62 and (2) questionnaires administered to 225 individuals ages 19-80 and older at four Virginia Department of Motor Vehicles offices to determine the most effective message content and format for a pedestrian signal education placard. The newly developed placard was installed at six intersections in Virginia, Maryland, and New York. Observational studies of more than 4,300 pedestrians during 600 signal cycles found no change in pedestrian signal compliance. However, results from questionnaires administered to 92 subjects at Departments of Motor Vehicles in Virginia, Maryland, and New York indicated a significant increase in understanding of the phases of the pedestrian signal. The authors concluded that although pedestrian crossing behavior is more influenced by the presence or absence of traffic than the signal indication, the wording on the placard was based on quantitative procedures using a relatively large number of subjects and should be used where signal educational placards are installed. The wording of the educational placard recommended by Knoblauch, Nitzburg, Reinfurt, et al. (1995) is shown in Recommendation 3 of Design Element P. A modification for a two-stage crossing is shown in Recommendation 4.

Zegeer and Cynecki (1986) tested a LOOK FOR TURNING VEHICLES pavement marking in a crosswalk, as a low-cost countermeasure to remind pedestrians to be alert for turning vehicles, including right-turn-on-red (RTOR) vehicles. Results showed an overall reduction in conflicts and interactions for RTOR vehicles and also for the total number of turning vehicles. Even with an RTOR prohibition, approximately 20 percent of motorists committed an RTOR violation when given the opportunity (Zegeer and Cynecki, 1986). Of those violations, about 23.4 percent resulted in conflicts with pedestrians or vehicles on the side street.

Zegeer, Opiela, and Cynecki (1982) conducted a crash analysis to determine whether pedestrian crashes are significantly affected by the presence of pedestrian signals and by different signal timing strategies. They found no significant differences in pedestrian crashes between intersections that had standard-timed (concurrent walk) pedestrian signals compared with intersections that had no pedestrian signals. Concurrent or standard timing provides for pedestrians to walk concurrently (parallel) with traffic flow on the WALK signal. Vehicles are generally permitted to turn right (or left) on a green light while pedestrians are crossing on the WALK interval. Other timing strategies include early release timing, late release timing, and exclusive timing. In early release timing--also termed a "leading pedestrian interval"--the pedestrian WALK indication is given before the parallel traffic is given a green light, allowing pedestrians to get a head start into the crosswalk before vehicles are permitted to turn. In late release timing, the pedestrians are held until a portion of the parallel traffic has turned. Exclusive timing is a countermeasure where traffic signals are used to stop motor vehicle traffic in all directions simultaneously for a phase each cycle, while pedestrians are allowed to cross the street. "Barnes Dance" or "scramble" timing is a type of exclusive timing where pedestrians may also cross diagonally in addition to crossing the street. Exclusive timing is intended to virtually eliminate turning traffic or other movements that conflict with pedestrians while they cross the street. In the Zegeer et al. (1982) analysis, exclusive-timed locations were associated with a 50 percent decrease in pedestrian crashes for intersections with moderate to high pedestrian volumes when compared with both standard-timed intersections and intersections that had no pedestrian signals. However, this timing strategy causes excessive delays to both motorists and pedestrians. Older road users (age 65 and older) recommended the following pedestrian-related countermeasures for pedestrian signs and signals, during focus group sessions held as a part of the research conducted by Knoblauch, Nitzburg, Reinfurt, et al. (1995): (1) reevaluate the length of pedestrian walk signals due to increasingly wider highways, (2) implement more Barnes Dance signals at major intersections, and (3) provide more YIELD TO PEDESTRIANS signs in the vicinity of heavy pedestrian traffic.

Several studies have been conducted to determine whether regulatory signing aimed at turning motorists could reduce conflicts with pedestrians. Zegeer, Opiela, and Cynecki (1983) found that the regulatory sign YIELD TO PEDESTRIANS WHEN TURNING was effective in reducing conflicts between turning vehicles and pedestrians. They recommended that this sign be added to the MUTCD as an option for use at locations with a high number of pedestrian crashes involving turning vehicles. Zegeer and Cynecki (1986) found that the standard NO TURN ON RED sign with the supplementary WHEN PEDESTRIANS ARE PRESENT message was effective at several sites with low to moderate right-turn vehicle volumes. However, it was less effective when RTOR volumes were high. It was therefore recommended that the supplemental message WHEN PEDESTRIANS ARE PRESENT be added to the MUTCD as an accepted message that may be used with an NTOR sign when right-turn volume is light to moderate and pedestrian volumes are light or occur primarily during intermittent periods, such as in school zones. The supplemental message when added to the NTOR sign with the circular red symbol reduced total pedestrian conflicts at one site and increased RTOR usage (as desired, from 5.7 percent to 17.4 percent), compared with full-RTOR prohibitions. It was recommended that the supplemental message be added to the MUTCD for the NTOR sign with the circular red symbol, under low to moderate right-turn vehicle volumes and light or intermittent pedestrian volumes.

More recently, Abdulsattar, Tarawneh, and McCoy (1996) found that the TURNING TRAFFIC MUST YIELD TO PEDESTRIANS sign was effective in significantly reducing pedestrian-vehicle conflicts during right turns. The sign was installed at six marked crosswalks in Nebraska, where right-turn vehicle-pedestrian conflict data were collected before and after its installation in an observational field study. For the six study crosswalks combined, a conflict occurred in 51 percent of the observations in the before period, but in only 38 percent of the observations during the after period. The reductions in pedestrian-vehicle conflicts across the observation sites ranged from 15 to 30 percent, and were statistically significant.

Turning toward a consideration of pedestrian walking times, section 4E.09 of the MUTCD (2000) indicates that a pedestrian clearance interval shall be provided immediately following the WALK indication, and should consist of a flashing DON'T WALK interval of sufficient duration to allow a pedestrian crossing in the crosswalk to leave the curb and travel at a normal walking speed of 1.2 m/s (4.0 ft/s) to at least the center of the farthest traveled lane, or to a median, before opposing vehicles receive a green indication. The MUTCD further states that, "where pedestrians who walk slower than normal or pedestrians who use wheelchairs routinely use the crosswalk, a walking speed of less than 1.2 m should be considered in determining the pedestrian clearance time."

Older pedestrian walking speed has been studied by numerous researchers. ITE (1999) reports walking speeds obtained by Perry (1992) for physically impaired pedestrians. Average walking speeds for pedestrians using a cane or crutch were 0.80 m/s (2.62 ft/s); for pedestrians using a walker, 0.63 m/s (2.07 ft/s); for pedestrians with hip arthritis, 0.68 to 1.11 m/s (2.24 to 3.66 ft/s); and for pedestrians with rheumatoid arthritis of the knee, 0.75 m/s (2.46 ft/s). Sleight (1972) determined that there would be safety justification for use of walking speeds between 0.91 and 0.99 m/s (3.0 to 3.25 ft/s), based on the results of a study by

Sjostedt (1967). In this study, average adults and the elderly had walking speeds of 1.37 m/s (4.5 ft/s); however, 20 percent of the older pedestrians crossed at speeds slower than 1.2 m/s (4.0 ft/s). The 85th percentile older pedestrian walking speed in that study was 1.04 m/s (3.4 ft/s). A 1982 study by the Minnesota Department of Transportation found that the average walking speed of older pedestrians was 0.91 m/s (3.0 ft/s). In a study conducted in Florida, it was found that a walking speed of 0.76 m/s (2.5 ft/s) would accommodate 87 percent of the older pedestrians observed (ITE, undated). Weiner (1968) found an average rate for all individuals of 1.29 m/s (4.22 ft/s), and of 1.13 m/s (3.7 ft/s) for women only. A Swedish study by Dahlstedt (undated), using pedestrians age 70 and older, found that the 85th percentile comfortable crossing speed was 0.67 m/s (2.2 ft/s).

Interviews and assessments were conducted with 1,249 persons age 72 and older from the New Haven, CT community of Established Populations for Epidemiologic Studies of the Elderly, to determine walking speeds and self-reported difficulty with crossing the street as pedestrians (Langlois, Keyl, Guralnik, Foley, Marottoli, and Wallace, 1997). The study population excludes persons in nursing homes or hospitals. In a telephone interview, 11.4 percent indicated that they had difficulty crossing the street. Reasons provided included insufficient time to cross and difficulty with right-turning vehicles. The mean walking speed for those reporting difficulty crossing the street was 0.38 m/s (1.25 ft/s), and for those reporting no difficulty was 0.59 m/s (1.94 ft/s). Only 7.3 percent of the population had measured walking speeds 0.91 m/s (3 ft/s), and less than 1 percent had walking speeds of 1.2 m/s (4.0 ft/s).

Hoxie and Rubenstein (1994) measured the crossing times of older and younger pedestrians at a 21.85-m (71.69-ft) wide intersection in Los Angeles, CA, and found that older pedestrians (age 65 and older) took significantly longer than younger pedestrians to cross the street. In this study, the *average* walking speed of the older pedestrians was 0.86 m/s (2.8 ft/s), with a standard deviation of 0.17 m/s (0.56 ft/s); the average speed of the younger pedestrians was 1.27 m/s (4.2 ft/s), with a standard deviation of 0.17 m/s (0.56 ft/s). Of the 592 older pedestrians observed, 27 percent were unable to reach the curb before the light changed to allow cross traffic to enter the intersection, and one-fourth of this group were stranded at least a full traffic lane away from safety. A study of crossing speeds by Coffin and Morrall (1995) limited to 15 pedestrians age 60 or older, at each of 6 crosswalk locations in Calgary, Canada, documented an 85th percentile walking speed of 1.0 m/s (3.28 ft/s) for midblock crosswalks and 1.2 m/s (4.0 ft/s) for crosswalks at signalized intersections. The authors note that the walking speed of older pedestrians varies according to functional classification, gender, and intersection type, and stated that approximately 95 percent of pedestrians in this study would be accommodated using a design walking speed of 0.8 m/s (2.62 ft/s).

Much more extensive observations of pedestrian crossing behavior were conducted at two crosswalk locations at two intersections in Sydney, Australia (a major 6-lane divided street, and a side street), where the design crossing speed was changed from 1.2 m/s to 0.9 m/s (4.0 ft/s to 3.0 ft/s) (Job, Haynes, Quach, Lee, and Prabhaker, 1994). Observations were made during 3,242 crossings during a baseline period (1.2 m/s [4.0 ft/s] design crossing speed) and 2 and 6 weeks after the flashing DON'T WALK interval was extended to allow for the slower crossing speed under study. This study was conducted to evaluate countermeasures to address the over-representation of pedestrians age 70 and older in crashes in the greater Sydney metropolitan area. At all crosswalk locations, the WALK phase remained a constant 6 s, and the clearance interval was extended from 14 s to 20 s at one intersection 18.2 m (59.7 ft) wide, and from 18 to 20 s at the other intersection measuring 24.2 m (79.4 ft) wide. Observations were conducted for 2,377 pedestrians ages 20-59, 511 pedestrians ages 60-65, and 354 pedestrians age 66 and older. The number of males and females was approximately equal. For both intersections, a general trend showed that the older the pedestrian, the longer the crossing time. Also, females crossed more slowly than males in all age groups. At the wider intersection, mean crossing speeds were 1.5 m/s (4.9 ft/s) for pedestrians ages 20-59; 1.3 m/s (4.27 ft/s) for pedestrians ages 60-65, and 1.1 m/s (3.6 ft/s) for pedestrians age 66 and older. The mean walking speed for females age 66 and older was 1.0 m/s (3.28 ft/s). The authors note that the assumed walking speed of 1.2 m/s (4.0 ft/s) leaves almost 15 percent of the total population walking below the assumed speed. Extending the clearance interval resulted in a decrease in the percentage of pedestrian-vehicle conflicts, from 4 percent in the baseline period to 1 percent in the experimental period at 2 weeks and also 1 percent at 6 weeks, at the wider intersection. This difference was significant at the p.001 level. Observed changes in pedestrian-vehicle conflicts at the smaller intersection were contaminated by an increase in the proportion of pedestrians (in the young and young/middle age groups only) who crossed illegally (began to cross during the flashing DON'T WALK phase); consequently, sustained differences between the baseline and experimental phases were not demonstrated. At the conclusion of this research, the authors recommended a reduction in the design walking speed from 1.2 m/s to 0.9 m/s (4.0 ft/s to 3.0 ft/s) at locations where there is significant usage by older pedestrians.

Knoblauch, Nitzburg, Dewar, et al. (1995) conducted a series of field studies to quantify the walking speed, start-up time, and stride length of pedestrians younger than age 65 and pedestrians 65 and older under varying environmental conditions. Analysis of the walking speeds of 3,458 pedestrians younger than age 65 and 3,665 pedestrians age 65 and older crossing at intersections showed that the mean walking speed for younger pedestrians was 1.51 m/s (4.95 ft/s) and for older pedestrians was 1.25 m/s (4.11 ft/s). The 15th percentile speeds were 1.25 m/s and 0.97 m/s (4.09 ft/s and 3.19 ft/s) for younger and older pedestrians, respectively. These differences were statistically significant. Among the many additional findings with regard to walking speed were the following: pedestrians who start on the WALK signal walk slower than those who cross on either the flashing DON'T WALK or steady DON'T WALK; the slowest walking speeds were found on local streets while the faster walking speeds were found on collector-distributors; sites with symbolic pedestrian signals had slower speeds than sites with word messages; pedestrians walk faster where RTOR is not permitted, where there is a median, and where there are curb cuts; faster crossing speeds were found at sites with moderate traffic volumes than at sites with low or high vehicle volumes.

For design purposes, a separate analysis was conducted by Knoblauch, Nitzburg, Dewar, et al. (1995) for pedestrians who complied with the signal, as they tended to walk more slowly than those who crossed illegally. The mean crossing speed for the young compliers was 1.46 m/s (4.79 ft/s) and for the older compliers was 1.20 m/s (3.94 ft/s). The 15th percentile speed for the young compliers was 1.21 m/s (3.97 ft/s) and was 0.94 m/s (3.08 ft/s) for the older compliers. Older female compliers showed the slowest walking speeds, with a mean speed of 1.14 m/s (3.74 ft/s) and a 15th percentile of 0.91 m/s (2.97 ft/s). One of the slowest 15th percentile values (0.89 m/s [2.94 ft/s]) was observed for older pedestrians crossing snow-covered roadways. It was concluded from this research that a mean design speed of 1.22 m/s (4.0 ft/s) is appropriate, and where a 15th percentile is appropriate, a walking speed of 0.91 m/s (3.0 ft/s) is reasonable. It was also determined by Knoblauch, Nitzburg, Dewar, et al. (1995) that the slower walking speed of older pedestrians is due largely to their shorter stride lengths. The stride lengths of all older pedestrians are approximately 86 percent of those of younger pedestrians.

Knoblauch, Nitzburg, Dewar, et al. (1995) also measured start-up times for younger and older pedestrians who stopped at the curb and waited for the signal to change before starting to cross. The mean value for younger pedestrians was 1.93 s compared with 2.48 s for older pedestrians. The 85th percentile value of 3.06 s was obtained for younger pedestrians, compared with 3.76 s for older pedestrians. For design purposes, the authors concluded that a mean value of 2.5 s and an 85th percentile value of 3.75 s would be appropriate. These data specifically did not include pedestrians using a tripod cane, a walker, or two canes; people in wheelchairs; or people walking bikes or dogs. The MUTCD (2000) states that the WALK interval should be at least 7 s long so that pedestrians will have adequate opportunity to leave the curb or shoulder before the pedestrian clearance time begins (except where pedestrian volumes and characteristics do not require 7 s, a 4-s interval may be used). Parsonson (1992) noted that the reason this much time is needed is because many pedestrians waiting at the curb watch the traffic, and not the signals. When they see conflicting traffic coming to a stop, they will then look at the signal to check that it has changed in their favor. If they are waiting at a right-hand curb, they will often take time to glance to their left rear to see if an entering vehicle is about to make a right turn across their path. Parsonson reported that a pedestrian reasonably close to the curb and alert to a normal degree can be observed to require up to 4 or 5 s for this reaction, timed from when the signal changes to indicate that it is safe to cross, to stepping off the curb. It may be remembered that older pedestrians stand farther away from the curb, and may or may not be alert. In addition, there are many drivers who run the amber and red signals, and it is prudent for pedestrians to "double-check" that traffic has indeed obeyed the traffic signal, and that there are no vehicles turning right on red or (permissive) left on green before proceeding into the crosswalk. Because older persons have difficulty dividing attention, this scanning and decision-making process requires more time than it would for a younger pedestrian. Parsonson (1992) reported that the State of Delaware has found that pedestrians do not react well to the short WALK and long flashing DON'T WALK timing pattern. They equate the flashing with a vehicle yellow period. The Florida Department of Transportation and the city of Durham, Ontario, provide sufficient WALK time for the pedestrian to reach the middle of the street, so that the pedestrian will not turn around when the flashing DON'T WALK begins.

One strategy that in recent implementations has appeared to offer promise in assisting pedestrians who are slower or more reluctant to cross when there is a perceived likelihood of conflict with turning vehicles is the leading pedestrian interval (LPI). A LPI is a brief, exclusive signal phase dedicated to pedestrian traffic. Van Houten, Retting, Farmer, and Van Houten (1997) investigated the effects of a 3-s LPI on pedestrian behavior and conflicts with turning vehicles at three urban intersections in St. Petersburg, FL. In the study, pedestrian-vehicle conflicts were observed during a baseline period, where the signal phasings at each intersection provided the onset of the pedestrian WALK signal and the onset of the green signal for turning vehicles concurrently. During the experimental phase, a 3-s LPI was installed to release pedestrian traffic three seconds before turning vehicles. The LPI was implemented using a modified, solid-state plug-in signal load switch that had the capacity to delay the change of the traffic signal phase from red to green. Pedestrians estimated to be age 65 and older were scored separately from those estimated to be age 12 and older. A total of 1,195 seniors and 3,680 nonseniors were observed across all three sites during the baseline condition. During the LPI condition, 860 seniors and 4,288 nonseniors were observed.

Observers collected data between 8:30 a.m. and 5:00 p.m., and scored the number of pedestrians who left the curb within 2 s before the start of the WALK indication, within 3 s after onset of the WALK indication, during the remainder of the WALK cycle, and during the flashing DON'T WALK indication. The number of conflicts was scored for each of these intervals, defined as any situation where a driver engaged in abrupt braking or either the driver or pedestrian took sudden evasive action to avoid a collision. Conflicts were scored separately for right-tuning and left-turning vehicles. Other data of interest included the number of times that a pedestrian yielded to a turning vehicle by stopping or waving the vehicle through, and the distance covered by the pedestrian during the LPI condition. The intersection geometries included the following: (1) one-way traffic with four northbound lanes by two-way traffic with one lane in each direction and diagonal parking (north and west crosswalks were observed because both included left-on-green conflict potential); (2) one-way traffic with four southbound lanes by two-way traffic with one lane in each direction and diagonal parking (south and east crosswalks were observed because both included left-on-green conflict potential); and (3) two-way traffic with two lanes in each direction by two-way traffic with two lanes in each direction (all four crosswalks were observed).

The number of conflicts per 100 pedestrians who started crossing during a defined 5-s begin-walk period (which began 2 s before and ended 3 s after the onset of the WALK indication) showed that during the baseline period, the number of conflicts averaged 3.0, 2.1, and 3.3 for the three sites. After the introduction of the LPI, the number of conflicts averaged 0.1, 0.1, and 0.2 for the three sites. The likelihood of conflict was significantly lower during the LPI condition than during the baseline condition for both left- and right-turning vehicles; the odds of conflict for pedestrians leaving the curb during the begin-walk period were reduced by approximately 95 percent. The reduction in odds conflict for seniors as a function of an LPI phase (89 percent reduction) was not significantly different from that of their younger counterparts (97 percent reduction). There was no significant effect of LPI on the odds of conflict for pedestrians leaving the curb *after* the begin-walk period, indicating that an LPI does not move conflicts to a later phase in the WALK interval.

The LPI also had the effect of significantly reducing the number of pedestrians yielding to turning vehicles; the odds of a pedestrian yielding to a turning vehicle were reduced by approximately 60 percent. Van Houten et al. (1997) indicate that once pedestrians were in the crosswalk, drivers acknowledged their presence and were more likely to yield the right of way. Also, they state that pedestrians occupying the crosswalk were more visible to drivers who were waiting for the light to change than they would have been had the drivers and pedestrians been released concurrently. The final measure of interest was the mean distance traveled by the lead pedestrian during the LPI condition, which averaged 2.6 m (8.5 ft). The authors state that this distance (which is greater than one-half of a lane width) appears sufficient for pedestrians to assert their right of way ahead of turning vehicles, and reduces conflicts that may result when pedestrians and vehicles begin to move at the same time.

Q. Design Element: Roundabouts

Table 26. Cross-references of related entries for roundabouts.

Applications in Standard Reference Manuals			
MUTCD (2000)	**Highway Capacity Manual (1998)**	**Roundabouts: An Informational Guide (2000)**	
Sects. 3B.16 & 3B.24 Figs. 3B-26 & 3B-27	Pg. 10-81, Items 3 & 7-10 Pg. 10-82, Fig. 10-37	Pgs. 1-5 & 1-6, Exhibits 1-3 & 1-4 (*Entry Width*, *Circulatory Roadway Width,* & *Inscribed Circle Diameter*) Pg. 1-7, Exhibit 1-5, Items c & d Pg. 1-11, Exhibit 1-6, Item d Pg. 1-13, Exhibit 1-7 Pg. 1-16, Sect. 1.6.4 Pg. 1-18, Sect. 1.6.6 Pg. 2-1, Exhibit 2-1 Pgs. 2-3 through 2-5, Sect. 2.1.1.1 Pg. 2-10, Para. 3 Pg. 2-13, Para. 6	Pg. 5-1, 1st Bullet Pgs. 5-2 - 5-4, Sect. 5.2.11 Pg. 5-7, Para. 2 Pg. 5-8, Para. 1 Pg. 5-10, Exhibit 5-9 Pg. 5-15, Para. 1 Pg. 5-19, Para. 3 Pg. 5-20, *Entry-Circulating* Equation Pg. 5-21, 1st & 2nd Bullets Pgs. 6-5 & 6-6, Sect. 6.2.1.3 Pgs. 6-17 through 6-21, Sects. 6.3.1, 6.3.2 & 6.33.1 Pg. 6-22, Para. 3 Pgs. 6-26 - 6-28, Sects. 6.3.7 & 6.3.8

		Pg. 2-17, Para. 3 Pg. 3-1, Bullet *Steps 2* & *5* Pg. 3-3, Last two bullets on pg. Pg. 3-5, Sects. 3.3 & 3. 3.1 Pg. 3-11, Item 3 Pgs. 3-19 & 3-20, Para. 2 on 1st pg. & Exhibits 3-14 & 3-16 Pgs. 4-1 & 4-2, Sect. 4.1.1 Pgs. 4-5 & 4-6, Paras. 3 & 5 on 1st pg. & Sect. 4.3.1 Pgs. 4-7 - 4-9, Sects. 4.3.4 & *4.3.5*	Pgs. 6-47 & 6-48, Sects. 6.5.2 & 6.5.3 Pg. 6-51, Para. 2 Glossary: *Central Island, Circulatory Roadway Width, Entry Width, Inscribed Circle Diameter, Pedestrian Refuge,* & *Splitter Island*

Countermeasures that have been suggested to reduce the occurrence of older driver crashes at intersections have included changes to intersection operations (e.g., protected left-turn phases, elimination of RTOR, redundant signing, etc.) and geometric design (e.g., full positive offset of opposite left-turn lanes, increases in turning radius for right turns, etc.). One proposed solution to reduce not only the frequency but also the severity of crashes at intersections is the installation of a modern roundabout (Harkey, 1995; Jacquemart, 1998). This countermeasure, it has been suggested, addresses problems that older drivers experience in judging speeds and gaps, understanding operational rules at complex intersections, and maneuvering through turns. Specifically, the following advantages of roundabouts for older road users have been postulated:

- Reductions in the speed of vehicles entering the intersection/circle-- this makes it easier to choose an acceptable gap to merge into, removes the need to accelerate quickly which occurs after a conventional right turn, and results in lower severity crashes with less serious injuries.
- The left turn is completely eliminated.
- The larger curb radius improves maneuverability.
- Simplified decision process results from one-way operation, yield-at-entry, and a reduced number of conflict points compared to a conventional intersection.
- A potential for improved pedestrian safety results from shorter crossing distances, fewer possibilities for conflicts with vehicles, and lower vehicle speeds--but, there are many unresolved issues surrounding the use of these facilities by (elderly and visually impaired) pedestrians at this time.

At the same time, there are significant human factors concerns about special driving task demands associated with the geometric and operational characteristics of roundabouts, and their novelty in this country. First, the driver approaching a roundabout must comprehend the prescribed movements, and in particular the yield-on-entry operation, as conveyed by upstream signing. For some years to come, these TCD's will be novel to motorists; and older persons are at a disadvantage in responding to novel, unexpected stimuli. Upon closer approach, the appropriate speed and heading changes to conform to the splitter island's controlling channelization must be performed; and where increased crash experience has been documented following roundabout installation, as discussed below, excessive entry speeds have been the prevalent contributing factor. Again, vehicle control for smooth entry may be more challenging for older than for younger drivers. At the point of entry, depending upon the deflection angle of the splitter island, there are critical seconds where confirmation that no conflict exists with a vehicle already in the roundabout requires a glance orientation that well exceeds 90. The increased difficulty for older drivers for visual search at skewed intersections has been underscored elsewhere in this Handbook (see page 69).

During negotiation of a roundabout, the ability to share attention between path guidance; gap (headway) maintenance; and visual detection, recognition, comprehension, and decision making associated with exit location cues is a near-continuous requirement, even for single-lane facilities. With multiple lanes, the avoidance of conflicts with adjacent vehicles places an exaggerated demand on motorists' attention-sharing abilities; and of course, the increased traffic volumes and speeds associated with these higher-capacity installations pose still greater demands. In the absence of controlled studies in the use of roundabouts by older drivers, it can only be stated qualitatively that information processing capacity will be exceeded sooner for older than younger persons, and that accommodation by some seniors--probably by reducing their speed while in the roundabout--is likely. This will detract from the operational benefits roundabouts are designed to produce, and may impact safety as well.

A better understanding of the operational and safety issues surrounding the use of roundabouts by older drivers and pedestrians depends upon crash data analyses from the limited number of existing facilities, and controlled and observational research in this area. This will require time, and more and more of these facilities are expected to come into operation in the immediate future. Thus, recommendations about when and why to use roundabouts to accommodate older road users remain premature, but an understanding of roundabout task demands that pose special difficulty for seniors allow for certain recommendations regarding preferred practices when a jurisdiction has decided to install a roundabout. The recommendations presented for this design element attempt to balance the human factors considerations above with the accumulating body of information supporting roundabout usage, discussed below.

AASHTO does not maintain standards for the design of roundabouts; however, FHWA has recently developed a document entitled, *Roundabouts: An Informational Guide* (FHWA, 2000). The *Highway Capacity Manual* (1997) includes a proposed capacity formula for roundabouts. Presently, only several States have design guidelines for roundabouts (Florida, 1996 and Maryland, 1995) based largely on Australian guidelines. Both Florida and Maryland use SIDRA software (Australian methodology) to conduct an analysis of the capacity of a planned roundabout, which is available through McTrans at the University of Florida at Gainesville. A guide written for the California Department of Transportation by Ourston and Doctors (1995) is based on British standards; according to Jacquemart (1998), Caltrans decided not to publish it. However, California DOT has distributed a Design Information Bulletin (No. 80) to provide general guidance to project engineers on appropriate applications, site requirements, geometric elements, and traffic analysis. New York State is developing an *Engineering Instruction* (EI) on roundabouts that will base design guidance on British Guides and software (RODEL). The EI notes that other software programs are permitted (e.g., Highway Capacity, SIDRA, ARCADY), provided that a RODEL analysis is performed for comparison purposes. This EI is to provide interim guidance for current projects, and will be incorporated into the NY State Highway Design Manual.

Flannery and Datta (1996) indicate that roundabouts are commonly used in Australia, Great Britain, France, Germany, Denmark, Ireland, Norway, Portugal, Spain, South Africa, Sweden, Switzerland, and the Netherlands. Sarkar, Burden, and Wallwork (1999) state that modern roundabouts are gaining in popularity in cities across the U.S. (in Arizona, California, Colorado, Florida, Kansas, Maryland, Massachusetts, Nevada, Oregon, Texas, and Wisconsin) because of their success in reducing speeds and the number of collisions. Because speeds are reduced, crashes are less severe. Because perpendicular left and right turns are eliminated, a roundabout with one-lane entries has fewer potential conflict points than a conventional intersection (8 vehicle-to-vehicle conflicts and 8 vehicle-to-pedestrian conflicts for a roundabout with 4, 1-lane entries, compared to 32 vehicle-to-vehicle conflicts and 24 vehicle-to-pedestrian conflicts for a conventional four-leg intersection). Jacquemart (1998) reports that as of the middle of 1997, there were fewer than 50 modern roundabouts in the U.S., compared to more than 35,000 in the rest of the world, with France owning the leading number of roundabouts (15,000 modern roundabouts currently, and growing at a rate of 1,000 per year).

Flannery and Datta (1996) highlight the fact that modern roundabouts are different than earlier rotaries and traffic circles common in the early 1900's. First, the modern roundabout requires drivers who are entering the circle to yield to traffic already in the circle (known as "offside priority"). Early roundabout operations gave priority to drivers entering the circle ("nearside priority"), which caused circulating traffic to come to a complete stop resulting in grid-lock. As a result of nearside priority, Flannery and Datta state that the operational performance of traffic circles declined rapidly with the increase in traffic beginning in the 1950's. Because traffic engineers believed that the problem was increased volume as opposed to nearside priority, traffic circles were generally abandoned in the U.S. Studies conducted in the Netherlands, Victoria Australia, and Western Australia have found significant reductions in crashes and casualty rates (from 60 to 90 percent fewer) at roundabouts converted from the old priority to the yield-on-entry priority.

Two other improvements in modern roundabout design are *deflection*, which helps to slow entering vehicles, resulting in safer merges with the circulating traffic stream, and *flared approaches*, which helps to increase capacity by increasing the number of lanes on the approach (Flannery and Datta, 1996). Jacquemart (1998) describes deflection as: "No tangential entries are permitted and no traffic stream gets a straight movement through the intersection. Entering traffic points toward the central island, which deflects vehicles to the right, thus causing low entry speeds." The splitter island is the geometric feature that physically separates entering traffic from exiting traffic, and defines the entry angle, which deflects and slows entering traffic. Looking at flared approaches from the viewpoint of accommodating older driver needs for simplicity, one-lane approaches are likely to be easier to negotiate. In the NCHRP *Synthesis of Roundabout Practice in the United States*, Jacquemart (1998) notes that safety benefits of roundabouts (from studies in Australia and Europe) seem to be greatest for single-lane roundabouts in rural conditions. Generally, safety benefits are related to the reduced speed in the roundabouts, the simplification of conflict points, and the "increased responsibility caused by the slower motion and the need to concentrate and yield, as compared to driver behavior in signalized intersections" (Jacquemart, 1998).

As noted earlier, studies performed to date to evaluate the safety performance of roundabouts have not included driver age as a variable. Flannery and Datta (1996) conducted a safety analysis of six sites in

Florida, Maryland, and Nevada that were converted from conventional intersections with traditional control (1-way stop, 2-way stop, or signalized) to roundabouts. All six sites had one-lane entrances and only one lane of circulating traffic. Five roundabouts had a posted speed of 56 km/h (35 mi/h) and one had a posted speed of 72 km/h (45 mi/h). Four of the sites had four approaches and two sites had three approach legs. Crash data were collected for a period of 1 to 3 years before and after retrofitting the sites (depending on location). Results of chi-squared and normal approximation statistical tests indicated that crash frequencies were significantly reduced in the period after the sites were retrofitted as modern roundabouts. The sites were not stratified by ADT or previous type of traffic control, as the sample size was small; therefore particular crash reduction factors were not identified. However, quick inspection of the crash frequencies provided by site indicate that only the roundabout retrofitted from a signalized intersection showed an increase in crashes in the after period; the other five sites (1-way and 2-way stop controlled) showed decreases in crash frequency in the after period (in the range of 60 to 70 percent). Analyses could only be performed on crash frequencies by group (as opposed to site), because traffic volumes before and after were not characterized, and the six retrofitted roundabouts varied in ADT from 4,069 to 17,825 vehicles.

Rahman (1995) and Jacquemart (1998) provided before and after crash data for the roundabout established in Lisbon, MD in 1993. In the six years prior to the roundabout, there were 45 reported intersection crashes with an average of eight crashes per year. From 1993 to 1995 (after roundabout installation), there were only two reported crashes. Before the roundabout, the crashes were almost all angle crashes, and after the roundabout was installed, one of the crashes was a single-vehicle crash against a fixed object, and the other crash was a rear-end crash. Injury crashes decreased from 4.3 per year to 0.3 Total delays decreased by 45 percent, from 1.2 vehicle hours to 0.34 vehicle hours in the morning peak hour and from 1.09 vehicle hours to 0.92 vehicle hours in the afternoon peak. This roundabout has four approach legs; it was retrofitted from a 2-way stop-controlled (flashing red beacon) intersection. The ADT was 8,500 vehicles (in March of 1995). The inscribed diameter is 30.5 m (100 ft); there are one-lane entries measuring 5.5 m (18 ft); there is one lane of circulating traffic that is 5.5-m (18-ft) wide; and in 1995 the peak hour total approach volume was 630 (Jacquemart, 1998). Rahman (1995) states that, "the performance of this first experimental roundabout in Maryland demonstrates the safety of roundabouts when properly designed."

Jacquemart (1998) examined the before and after crash data of 11 roundabouts in the U.S. Results are described for large roundabouts with three-lane entries (one in Long Beach, CA and two in Vail, CO) and smaller roundabouts with one- or two-lane entries and inscribed circle diameters of 37 m (121 ft) or less (Santa Barbara, CA; Lisbon, Cearfoss, Lothian, and Leeds, MD; Tampa, FL; Montpelier, VT; and Hilton Head, SC). He states that the small- to moderate- size roundabouts showed significant reductions in total crashes (from an average annual crash frequency 4.8 to 2.4, or 51 percent) and injury crashes (from an average annual crash frequency of 2.0 to 0.5, or 73 percent). There were no statistically significant differences in property-damage-only (PDO) crashes at the smaller roundabouts, although there was a reduction from 2.4 to 1.6 average annual crashes, or 32 percent. Although there was a trend toward crash reduction for the larger roundabouts, there were no statistically significant reductions in total crashes, injury crashes, or PDO crashes. Each roundabout experienced a reduction in injury crashes ranging from 20 to 100 percent. PDO crashes increased at a roundabout in Vail, CO from 15 to 18 per year, and at Leeds, MD from 1.5 to 5.3 per year. At the other 9 roundabouts, however, PDO crashes decreased from 6 to 1 per year. Although PDO crashes at the Leeds, MD site showed an increase, injury crashes decreased from 2.2 to 0.0 per year. The PDO crashes at this site were all single-vehicle crashes that occurred because the vehicles entered the roundabout too fast. Jacquemart (1998) reports findings by Niederhauser, Collins, and Myers (1997) who showed that the average cost per crash decreased by 30 percent across the 5 conventional intersections in Maryland that were retrofitted to roundabouts, from $120,000 before the roundabout to $84,000 after the roundabout.

Niederhauser, Collins, and Myers (1997) reported the before and after average annual crash history for the five intersections in Maryland that were converted to roundabouts. All sites are single-lane approach and single-lane circulating roundabouts. Overall, the average crash rate was reduced from an average of 5.0 crashes per year to an average of 2.4 crashes per year, which is a reduction of greater than 50 percent. Data for each roundabout is reported in table 27.

Persuad, Retting, Garder, and Lord (2000) evaluated the change in crashes following conversion of 24 intersections in urban, suburban, and rural environments in 8 States (CA, CO, FL, KS, ME, MD, SC, and VT) from stop-sign or signal control to modern roundabouts. The Bayes procedure was used to account for regression to the mean and to normalize differences in traffic volume between the before and after periods. The number of months of crash data available in the before period ranged from 21 to 66, and the number of months of crash data available in the after period ranged from 15 to 68. Across all sites and crash severities, crashes were reduced by 39 percent in the after-conversion period. A 76-percent reduction was estimated in the after period for injury crashes. For the 20 sites where injury data were available, there were 3 fatal crashes in the before period, and none in the after period. There were 27 incapacitating injury crashes in the before period, and 3 in the after period. Thus, the estimated reduction in fatal and incapacitating injury crashes is 89 percent.

Table 27. Before and after average annual crash history for the five intersections in Maryland that were converted to roundabouts.
Source: Niederhauser, Collins, and Myers, 1997.

Site	Average Annual Crashes	
	Before	After
Lisbon	6.0	2.0
Cearfoss	2.7	0
Leeds	3.3	4.9
Lothian	7.7	5.1
Taneytown	5.3	0

Persuad et al. (2000) looked at the crash reduction rates as a function of operating environment and before-conversion control. For the 9 urban single-lane roundabouts converted from stop control, a 61-percent reduction was estimated for all crash severities combined, and a reduction of 77 percent was estimated for injury crashes. For the 5 rural single-lane roundabouts converted from stop control, a 58-percent reduction was estimated for all crash severities combined, and a reduction of 82 percent was estimated for injury crashes. For the 7 urban multilane roundabouts, a 15 percent reduction in crashes of all severities was estimated. Injury data were not available for four these sites in the before-conversion period. For the 3 roundabouts converted from traffic signal control, all crashes were reduced by 32 percent, and injury crashes by 68 percent. The authors note that the smaller safety effects for the group of urban multilane roundabouts suggests that there may be differences in safety performance for single-lane designs compared to multi-lane designs. However, they caution that all seven of these roundabouts were located in one State (CO) where three of the four in Vail, CO are part of a freeway interchange that also includes nearby intersections that were previously four-way stop-controlled. Finally in this research, pedestrian and bicycle crash samples were too small to be meaningful, however, there were three reported pedestrian crashes during the before period and one with minimal injuries in the after period. Four bicyclists were injured in the before period and three during the after period.

Wallwork (1993) notes that crashes do occur at roundabouts, and consist of rear-end or merge-type crashes. Both crash types are low speed and low impact, and result in few-- if any--injuries. He states that with a roundabout, "no one can 'run the red,' and cause a right-angle collision, nor can drivers make a mistake in selecting a gap in the approaching through traffic when making a left turn. The only decision an entering driver needs to make is whether or not the gap in the approaching/circulating traffic is large enough to enter safely." Lower speeds (less than 40 km/h [25 mi/h]) results in shorter braking distances and longer decision making times. Even if a driver makes a mistake and chooses a gap that is too short, a collision is easier to avoid. Thus, the reduction in task difficulty coupled with the low speed environment, results in an overall reduction in the number of crashes, and a reduction in the severity of the crashes that do occur, which should be especially beneficial to older persons.

The delays before and after eight intersections (seven of which were two-way, or multi-way stop controlled, and one was signalized) were converted to roundabouts were also described by Jacquemart (1998). The total delay (stopped delay plus move-up time in queue) for eight U.S. roundabouts before retrofit was 13.7 s for morning peak time and 14.5 s for afternoon peak time. This compares to 3.1 s for morning and 3.5 s for afternoon peak times after conversion to roundabouts. Delays were thus reduced by 78 percent in morning peak periods, and by 76 percent in afternoon peak periods, after intersections were converted to roundabouts.

Jacquemart (1998) received information about the design of 38 roundabouts in the U.S., and presented data for four major geometric features: (1) inscribed circle diameter; (2) circulatory roadway width; (3) central island; and (4) entry widths. The *inscribed circle diameter* is defined as the circle that can be inscribed within the outer curbline of the circulatory roadway. Twenty-eight of the 31 roundabouts for which data were provided on this element have an inscribed circle diameter in the range of 30 to 61 m (98 to 200 ft), with the majority of these (11) ranging from 30 to 32.9 m (98 to 108 ft). Regarding *circulatory roadway width*, 43 percent of the cases are 4.5- to 5.5-m wide (15- to 18- ft wide); 21 percent are 6.0 - to 7.0-m wide (20- to 23-ft wide); 25 percent are 7.3- to 9.1-m wide (24- to 30-ft); and 11 percent are 10.7- to 11.0-m (35- to 36-ft) wide. Thus, 36 percent are at least 2 lanes wide. The *central island* can be raised or flush, or it can be raised with a sloping curb or drivable apron surrounding it. The truck apron is generally included in the central island diameter. Jacquemart repots that approximately 66 percent of the roundabouts for which data were provided have central islands greater than 9 m (30 ft) in diameter. Regarding *entry widths*, 59 percent of the reported cases have single-lane entries, 30 percent have two-lane entries, and 11 percent have three or more lane entry legs. Studies in other countries help to shed some light on the optimum design characteristics of modern roundabouts.

In the Jacquemart (1998) synthesis, a study by Brilon (1996) of 34 modern roundabouts in Germany concluded that 30 m (98 ft) seemed to be the ideal inscribed diameter for a single-lane roundabout. Brilon states that smaller diameters result in larger circulatory roadways which reduces the deflection. Additionally, truck aprons with a rougher pavement are recommended, so that the circulatory roadway remains 4- to 4.5-m (13- to 15-ft) wide. In a study of 83 roundabouts in France (Centre D'Etudes Techniques de L'Equipment de l'Ouest, 1986) in Jacquemart (1998), it is also concluded that roundabouts with smaller diameters have fewer crashes than larger roundabouts or those with oval circles. Their data indicate that the 13 roundabouts with inscribed diameters of <30 m had a crash frequency of 0.69 crashes per roundabout. This compares to 1.54 crashes per roundabout for the 11 roundabouts with inscribed diameters of 30 to 50 m; 1.58 crashes per roundabout for the 26 roundabouts with inscribed diameters of 50 to 70 m; 1.81 crashes per roundabout for the 16 roundabouts with inscribed diameters of 70 to 90 m; 3.80 crashes per roundabout for the 8 roundabouts with inscribed diameters of 90 m or greater; and 4.40 crashes per roundabout for the 9 oval roundabouts.

Splitter islands are another geometric feature of modern roundabouts. These are generally raised islands that are placed within a leg of a roundabout to separate entering and exiting traffic, and to deflect entering traffic. They also serve as a safety zone for pedestrians. Only one of the 38 roundabouts has painted (marked) splitter islands. The study conducted in Germany (Brilon, 1996, in Jacquemart, 1998) concluded that splitter islands are important to the safety of pedestrians, and should be 1.6- to 2.5-m (5- to 8-ft) wide, with pedestrian crossings located 4 to 5 m (13 to 16 ft) back from the circulating roadway. A study conducted in Switzerland by Simon and Rutz, 1988 (in Jacquemart, 1998) also concluded that the distance between the pedestrian crossing and the inscribed circle should be 5 m (16 ft) as greater distances do not increase pedestrian safety. They recommended the use of splitter islands with safety zones for pedestrians for crossings of more than 300 vehicles per hour. Wallwork (1999) states that a feature of roundabouts that makes them safer for pedestrian than conventional intersections, is that pedestrians walk *behind* the cars. He recommends moving the crosswalk back one car length from the yield line for each lane of entry (i.e., one car length for a one-lane entry, two car lengths for a two-lane entry, or three car lengths for a three-lane entry). Brilon (1996) recommended Zebra-striped crossings only when there were more than 100 pedestrians crossing during the peak hour. Maryland (DOT/SHA, 1995) normally places pedestrian crossings 6 to 7.6 m (20 to 25 ft) from the yield line. Crosswalk striping is not used, to avoid driver confusion of crosswalk limit lines with yield lines. Special consideration is given in providing priority crossings for pedestrians where pedestrian volumes are high, where there is a high proportion of younger or older pedestrians, or where pedestrians experience particular difficulty in crossing, and are being delayed excessively. The agency believes that it is desirable to place these crossings at least 23 m (75 ft) downstream of the exit from the roundabout and possibly augment the crossing with a signal. This will reduce the possibility that vehicles delayed at the pedestrian crossing will queue back into the roundabout, and gridlock the whole intersection.

In the survey conducted by Jacquemart (1998) detailing 38 U.S. roundabouts, 56 percent of the sites were reported to have no or very few pedestrians, 22 percent have between 20 and 60 pedestrians during the peak hour, and 22 percent have more than 60 pedestrians per hour. Of particular interest is the Montpelier, Vermont roundabout, which is located next to a senior housing project and is also close to a middle school (400 students), and carries in excess of 260 pedestrians during each rush-hour (morning and afternoon) period on school days (Gamble, 1996; Redington, 1997). This roundabout has 3 legs, an inscribed diameter of 34 m, one-lane entries for each lane and one lane of circulating traffic. The AADT is approximately 11,000 (7,300 AADT for each leg) and carries approximately 40 tractor trailers (WB-62) each day (Redington, 1997). The peak hour total approach volume is 1,000 vehicles (Jacquemart, 1998). Prior traffic control was a one-way stop at a Y-intersection.

Jacquemart (1998) lists criteria to assist visually impaired pedestrians that include: (1) keeping the crossing away from the circle (e.g., 5 to 6 m from the outer circle) lets the blind person distinguish the exiting traffic from the circulating traffic; and (2) the splitter island provides a refuge where the pedestrian can shift his or her attention from one traffic stream to another. Different pavement texture for the walkways will assist the visually impaired pedestrian in locating the crosswalks. Drivers approaching a roundabout approach at speeds slower than they would for an approach to a conventional intersection; thus, they are more likely to stop for pedestrians, and may be more likely to notice a pedestrian on an approach to a roundabout because they are not concentrating on finding a gap in the opposing traffic stream to turn left.

Jacquemart (1998) also provided a summary of the current lighting, signing, and pavement marking practices at the 38 U.S. roundabouts for which questionnaire data were provided. First, all existing roundabouts were reported to have nighttime lighting. Next, all roundabouts were reported to have the standard

YIELD sign, although often it was supplemented by an additional plate with specific instructions, such as "TO TRAFFIC ON LEFT;" "TO TRAFFIC IN ROUNDABOUT;" or " TO TRAFFIC IN CIRCLE;" or with the international roundabout symbol, which is three arrows in a circular pattern. In addition, 90 percent of the roundabouts contain an advance YIELD AHEAD symbol sign and 7 percent use the YIELD AHEAD legend sign. Twenty-four percent included a supplemental plate on the advance YIELD sign that said "AT ROUNDABOUT," presented the roundabout symbol, or displayed a speed limit sign. All roundabouts had either a one-way sign (R6-1 or R6-2) or a large arrow warning sign (W1-6) in the central island. Chevron signs often accompanied the one-way signs (see figure 15). Regarding pavement markings, approximately 20 percent of the roundabouts supplemented the yield line at the roundabout entrance with the pavement marking legend "YIELD" or "YIELD AHEAD." For multilane roundabouts, only in the case of the Hilton Head, SC, roundabout were lane lines present.

Figure 15. One-Way and Chevron sign combination for use in central island of roundabout.

Jacquemart (1998) reported that the authorities responsible for the roundabout believe that the large number of senior drivers in the area would be more comfortable with lane markings in the circle. Simon and Rutz, 1988 (in Jacquemart, 1998) recommended that for main roads or national highways, advance directional signs with the roundabout symbol should supplement the roundabout yield sign at the entry, but that other special warning signs--such as roundabout ahead or priority to the left--are *not* recommended. Wallwork (1999) does not recommend the widespread use of supplemental signs (e.g., posting "TO TRAFFIC IN CROSSWALK" on the YIELD sign), because it constitutes visual clutter. Instead, he recommends their use only as a local measure to educate road users for a short time period after roundabout installation.

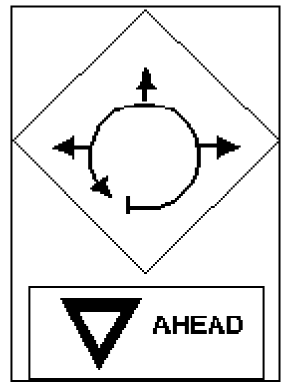

Figure 16. Signs used on approaches to Maryland roundabouts.

Maryland's practice (Maryland DOTSHA,1995) for State highway and county collector roads is to provide the following signs on the approach:

- Junction assembly.
- "Roundabout Ahead" warning signs, with Yield Ahead" plates (see figure 16).
- Destination guide signs (either conventional verbal signs with arrows or, for higher speed multi-lane approaches, the use of diagrammatic guide signs).
- "Yield Ahead" signs (W3-2A) in combination with Advisory speed plates (W13-1).
- Other guide signs, such as the Advance Route Marker Turn Assemblies.

At the roundabout intersection, the following signs are used:

- "Yield" (R1-2) signs in combination with "To Traffic in Circle"(see figure 17).
- "One Way" (R6-1R) signs in combination with obstruction markers (see figure 15).
- Exit guide signs.

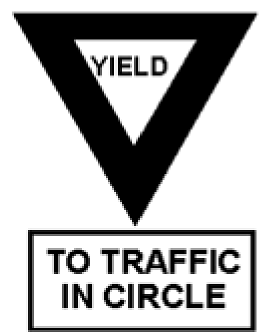

Figure 17. Signs used at Maryland roundabouts.

For local roadways, the following signs are recommended:

- "Roundabout Ahead" warning signs, with Yield Ahead" plates.
- Destination guide signs.
- "Yield" (R1-2) signs in combination with "To Traffic in Circle."
- "One Way" (R6-1R) signs in combination with obstruction markers.
- Exit guide signs with "Do Not Enter" (R5-1) mounted on the back.

Maryland's pavement markings at roundabouts consist of:

- A 200-mm to 400-mm (8-in to 16-in) wide yield line, with 0.91 m (3 ft) segments and 0.91 m (3 ft) gaps that marks the entrance to the roundabout.
- 400-mm (16-in) wide solid yellow hatch markings in the splitter island envelope.
- Raised retroreflective pavement markers delineating the splitter island envelope.
- A 200-mm yellow solid pavement marking delineating the inner circle.
- A 200-mm (8-in) wide white edgeline delineating the right side of the roadway from the beginning of the splitter island to the yield line and a 200-mm (8-in) wide yellow pavement marking delineating the splitter island envelope.
- Optional rumble strips to reduce approach speeds, usually for high-speed, rural approaches.

The use of lane lines in the circulating roadway, for multilane roundabouts is made on a case-by-case basis, as it is believed by the agency that pavement markings may confuse rather than assist drivers in negotiating the roundabout.

Jacquemart lists several location types where it is appropriate to install roundabouts, based on a review of guidelines from abroad and those existing guidelines in the U.S. (e.g., Maryland and Florida). These locations include:

- High crash locations, particularly with high crash rates related to cross movements or left- turn or right-turn movements.
- Locations with high delays.
- Four-way stop intersections.
- Intersections with more than four legs.
- Intersections with unusual geometry (Y or acute angle).
- Intersections with high left-turn flows.
- Intersections with changing traffic patterns.
- Intersections where U-turns are frequent or desirable along commercial corridors.
- At locations where storage capacities for signalized intersections are restricted, or where the queues created by signalized intersections cause operational or safety problems.
- Intersections where the character or speed of the road changes, such as at entry points to a community or at junctions where a bypass road connects to an arterial.

Ourston and Bared (1995) cited the work of Guichet (1992) who investigated 202 crashes at 179 urban roundabouts in France. The crash causes and relative frequencies are presented in table 28.

Table 28. Causes of crashes at urban roundabouts in France.
Source: Ourston and Bared, 1995

Cause of Crash	Percent of Crashes
Entering traffic failing to yield to circulating traffic	36.6
Loss of control inside the circulatory roadway	16.3
Loss of control at entries	10.0
Rear-end crashes at entries	7.4
Sideswipe, mostly at two-lane exits with cyclists (2 of 3 instances)	5.9
Running over pedestrians at marked crosswalks, mostly at two-lane entries	5.9
Pedestrians on the circulatory roadway	3.5
Loss of control at exits	2.5
Head-on collision at exits	2.5
Weaving inside the circulatory roadway	2.5

Guichet (1992) listed the major design recommendations, based on the findings of the crash investigation:

- Ensure that motorists recognize the approach to the roundabout.
- Avoid entries and exits with two or more lanes, except for capacity requirements.
- Separate the exit and entry by a splitter island.
- Avoid perpendicular entries or very large radii.
- Avoid very tight exit radii.
- Avoid oval-shaped roundabouts.

Wallwork (1999) recommends that in areas where there is a high concentration of senior drivers, it is desirable to use the lower end of the speed range that he has determined for roundabouts in a particular roadway class. He states that a roundabout meets drivers' requirements for simple decision making, and low speed is paramount for safe roundabout operation. His design-speed recommendations by roadway class are presented in table 29.

Table 29. Design-speed recommendations by roadway class, for modern roundabouts.
Source: Wallwork, 1999.

Roadway Classification	Roundabout Design Speed

Local Road	19-24 km/h (12-15 mi/h)
Collector Road	24-29 km/h (15-18 mi/h)
Secondary Arterial	29-34 km/h (18-21 mi/h)
Major Arterial	34-37 km/h (21-23 mi/h)
Rural Roadway	Maximum 40 km/h (25 mi/h)

He states that the best way to control driver behavior is through the use of concrete: the roundabout has a concrete circle in the center, which defines a path to control speed, and a roundabout uses concrete islands to deter wrong-way movements and to control entry speeds. Roundabouts that are not designed for slow speeds result in high crash rates; there are at least two in the U.S. (Boulder, CO and Daytona Beach, FL) that are being removed, because of poor design (e.g., no bulbouts for deflection on the entries allowing for 64 km/h [40 mi/h] speeds). One other feature of roundabouts that is important for all drivers, but older drivers in particular, is high visibility. Wallwork recommends that tall trees, fountains, or statues be placed in the center of the roundabout so that long-range vision (at least 152 m [500 ft] of preview distance) of the roundabout is available. This will let a driver know that a reduction in speed is necessary downstream.

Regarding public opinion about roundabout implementation, Taekratok (1998) indicates that people do not make a clear distinction between modern roundabouts and traffic circles, and therefore public responses to roundabout proposals are negative. Jacquemart (1998) presents copies of media coverage (*Howard County Sun* Newspaper) about the Lisbon, Maryland roundabout installed in Howard County as an experimental solution to an intersection with a high crash rate. One year before the roundabout opened, most of the Lisbon residents objected to the idea of a roundabout. Four months after the roundabout opened, a local citizen's committee voted overwhelmingly to make the roundabout permanent. Taekratok (1998) reports that the strategies taken by Florida, Maryland, and Vermont have been successful in improving public perception, and include public education through the use of brochures, videotapes, and mass media to provide information during the development stage. This will help the public to understand the differences between circles and roundabouts, and will gradually reduce opposition.

Redington (1997) notes that roundabouts are small (e.g., 28 to 55 m [91.8 to 180 ft]) compared to the old time traffic circles found in New England and New Jersey (e.g., 76 m [249 ft] or greater), and that drivers strongly dislike traffic circles with their typical operating speeds of 50 to 60 km/h (31 to 41 mi/h). While the Montpelier, VT, Keck Circle Roundabout was under construction, the Roundabout Demonstration Committee prepared educational materials that included a brochure providing safety rules for drivers and pedestrians, as well as news releases and public service announcements in response to negative public reaction during construction, and negative commentary from local morning radio personalities (Redington, 1997). This Committee also conducted a survey of 111 citizens working or living near the roundabout one year after its opening to measure public opinion. Of the 111 respondents, 104 had driven the roundabout, 89 had walked, and 19 had bicycled. "Very favorable" or "favorable" responses were obtained from 57.6 percent of the respondents, 27.9 percent of the responses were "neutral" and 14.4 percent were "unfavorable" or "very unfavorable." The survey contained two open-ended questions to allow respondents to contribute "likes," dislikes," and comments about "what they miss about the old intersection." The 111 respondents contributed 214 comments. The majority of the 65 "like" comments pertained specifically to smoother and better traffic movement. Fifty-six comments were obtained from respondents who "dislike" the roundabout. The majority of these were directed toward poor driver behavior such as drivers failing to yield, failing to follow the rules, and failure to use turn signals.

Finally, Sarkar, Burden, and Wallwork (in press) reviewed driver's manuals for 32 States and the District of Columbia, and concluded that the information on traffic circle and roundabout use was inadequate. Only 10 of the States provided some instruction in their manuals about how to use the circles (i.e., entering drivers should yield to drivers who are already in the circle) and none provided information about how to use roundabouts. Information about types of signs placed near roundabouts and circles was not present, nor was there any explanation about the differences between circles and roundabouts. Only one State had an illustration of a circle, but in the authors' opinion, it was not clear or easy to understand. They recommend that State driver manuals be revised to include information about correct use of traffic circles and roundabouts, as roundabouts are becoming increasingly popular in the U.S.

1. As per feedback provided by State engineers during a training workshop conducted by Handbook authors on August 6-7, 1998.

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Topics: Safety, Highway Design, Highway Operations, Driver Age, Driver Performance, Human Factors, Vision, Attention, Perception, Cognition, Memory, Physical Ability, Risk Perception, Hazard Perception
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TRT Terms: Roads--United States--Design and construction--Handbooks, manuals, etc, Older automobile drivers, Pedestrian facilities design--United States--Handbooks, manuals, etc, Roads--United States--Safety measures--Handbooks, manuals, etc, Roads--United States--Interchanges and intersections--Safety measures--Handbooks, manuals, etc, Aged drivers, Highway design, Human factors
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